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Design of the Distribution Portion for an Automated Closed Conduit Cut-back Furrow Irrigation System

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DESIGN OF THE DISTRIBUTION PORTION FOR AN AUTOMATED CLOSED CONDUIT CUT-BACK FURROW IRRIGATION SYSTEM

BY

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This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

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INTRODUCTION

At the present time, the utilization of water for irrigation purposes constitutes the largest single use of developed water in the United States. In 1965, 110,852 million gallons of water per day were used for irrigation purposes, while the projected estimation for the year 2000 is 149,852 million gallons per day (27). In addition to its present irrigation development, South Dakota is expecting an additional 450,000 acres to come under irrigation in this decade. This acreage will use approximately 1,600 million gallons per day in a normal 120-day growing season, based on 60 years of annual natural precipitation records.

As the need for irrigation water increases, so will the competition for available water. In 1960, the Department of Agriculture estimated the average irrigation efficiency of water use for surface irrigated farms in the United States to be 47 percent (14). This relatively low efficiency of water use stresses the need for improved irrigation methods and newly developed techniques if irrigation is to compete for the anticipated increase in demand for available water supplies.

Various methods of surface irrigation were used extensively during early irrigation development. Later, sprinkler irrigation methods were developed for use on land where topography or soil was unsuitable for surface methods. During the past few years, improved sprinkler equipment and automation of sprinkler systems have increased the demand for these new techniques.

Attempts to automate conventional surface irrigation systems have started only recently. The primary objective for automation of the surface irrigation system was to reduce labor requirements, but automation also improved management and control of the water with a resultant effect of an increase in efficiency. Haise and Kruse (15) conclude:

> One of the most promising possibilities to utilize water more efficiently on surface irrigated farms is to provide the farmer with irrigation labor-saving devices, for the irrigation farmer who automates to save labor should automatically save water providing he has a well-designed and maintained irrigation system.

The design of the newly developed surface irrigation systems has incorporated automation with water saving irrigation procedures. One irrigation method used is referred to as cut-back furrow irrigation. Cut-back furrow irrigation requires two rates of flow. An initial flow rate to wet the furrow length as rapidly as possible, without erosion, is followed by a smaller cut-back rate to match the soil intake rate of the furrow. This procedure generally increases application efficiency by reducing water runoff and deep percolation losses. The procedure also provides a more uniform application along the length of the field by decreasing the time differential for water intake from upper to lower portions of the furrow. Cut-back irrigation is not a new irrigation procedure. For many years those people interested in water conservation have suggested its use. Even with its apparent advantages, it has not been widely accepted because of the additional labor required to adjust flows when used without automation.

An automated closed conduit cut-back furrow irrigation system can reduce the amount of required labor as well as having the advantages of portability for ease of movement and flexibility for change when cropping practices are altered. While at the same time the system attains an increase in irrigation efficiency and application uniformity. Garton (9) states:

> A desirable system of automated irrigation from an engineering standpoint would be one which reduces labor the greatest amount, is economical, feasible, simple to build, operate, and maintain, and applies water uniformly and efficiently.

The design of the water distribution portion of an automated cut-back furrow irrigation system is the subject of this study.

SCOPE AND OBJECTIVES

The three major design areas of an automatic cut-back irrigation system can be divided into three categories, the water distribution portion, the automatic pressure regulating valves, and the automatic controls.

The area of concern of this study will be confined to the first category, the design of the distribution portion of the proposed furrow irrigation system.

Israelson (18) lists the ten major criteria of the well designed distribution system as follows:

1. Store the required water in the root zone of the soil.
2. Obtain reasonably uniform application of the water.
3. Minimize soil erosion.
4. Minimize runoff of irrigation water from the field.
5. Provide for beneficial use of the runoff water.
6. Minimize labor requirements for irrigation.
7. Minimize the land used for ditches and other controls to distribute the water.
8. Fit the irrigation system to the field boundaries.
9. Adapt the system to the soil and topographic changes.
10. Facilitate the use of machinery for farming practices used.

To enable a designer to fulfill this design criteria for a furrow cut-back system, several factors must be determined. The research presented herewith was undertaken to define more accurately the design components for the distribution system. The specific objectives of this research are as follows:

1. To investigate the discharge characteristics for nonregulating bored outlets.
2. To determine the factors influencing the uniformity of discharge of outlets spaced along the pipe.
3. To determine criteria for the design of the distribution banks.
4. To evaluate the operating characteristics of distribution banks under field conditions.

THE STATE OF THE ART

In response to the need for conservation of water, researchers have investigated many aspects of water use by irrigation. Particular emphasis has been placed on efficiency of water distribution rather than plant consumptive use. Automation of distribution systems is an avenue that is currently receiving widespread attention. Automation of sprinkler systems was given first recognition with the automation of surface methods being currently highlighted.

Bondurant and Humphreys (1) describe a completely automatic irrigation system as one which would determine when to irrigate, turn on the water, receive and distribute the water to all parts of the field, shut off the water when irrigation is completed, and reset the system. Such an automatic system would require properly prepared fields, controls for starting, regulating, and stopping the irrigation and a well designed distribution system.

Haise and others (16) have developed an automatic inflatable pneumatic valve for automating surface irrigation systems. The valve made of nylon reinforced butyl rubber is available in two shapes, the O-ring shape for use with alfalfa valves on buried pipelines and the lay-flat model for installation on open channel turnouts. When the valve is inflated, it expands shutting off the flow of water.

Deflation occurs when the air supply is removed and water pressure collapses the valve and irrigation begins.

Fischbach (6) has developed a completely automated furrow irrigation system incorporating a reuse system which returns the runoff water from the end of the field to the water supply line by means of an additional pumping unit installed in a catchment basin. The system includes an enclosed pipe system designed to carry water from the source to the point of release for irrigation. Additional components include: a method of sequencing irrigation sets, control of the water flowing over the land surface, a runoff collection and return system, soil moisture sensing devices which sense the need for water to start and stop the system, and electrical controls. By using a furrow stream size of three times the intake rate of the soil and returning the runoff to the supply line, tests showed that an application efficiency of 91.9 percent was attainable for a total water application of three inches. This compares to 64.8 percent when the reuse system was not included. The application uniformity coefficient was 91.8 (7).

Garton (10) has designed an automatic cut-back system for furrow irrigation. The system, shown schematically in Figure 1, was designed to deliver uniform flow to each furrow at both an initial and cut-back flow rate. The system consists of a series of horizontal, trapezoidal

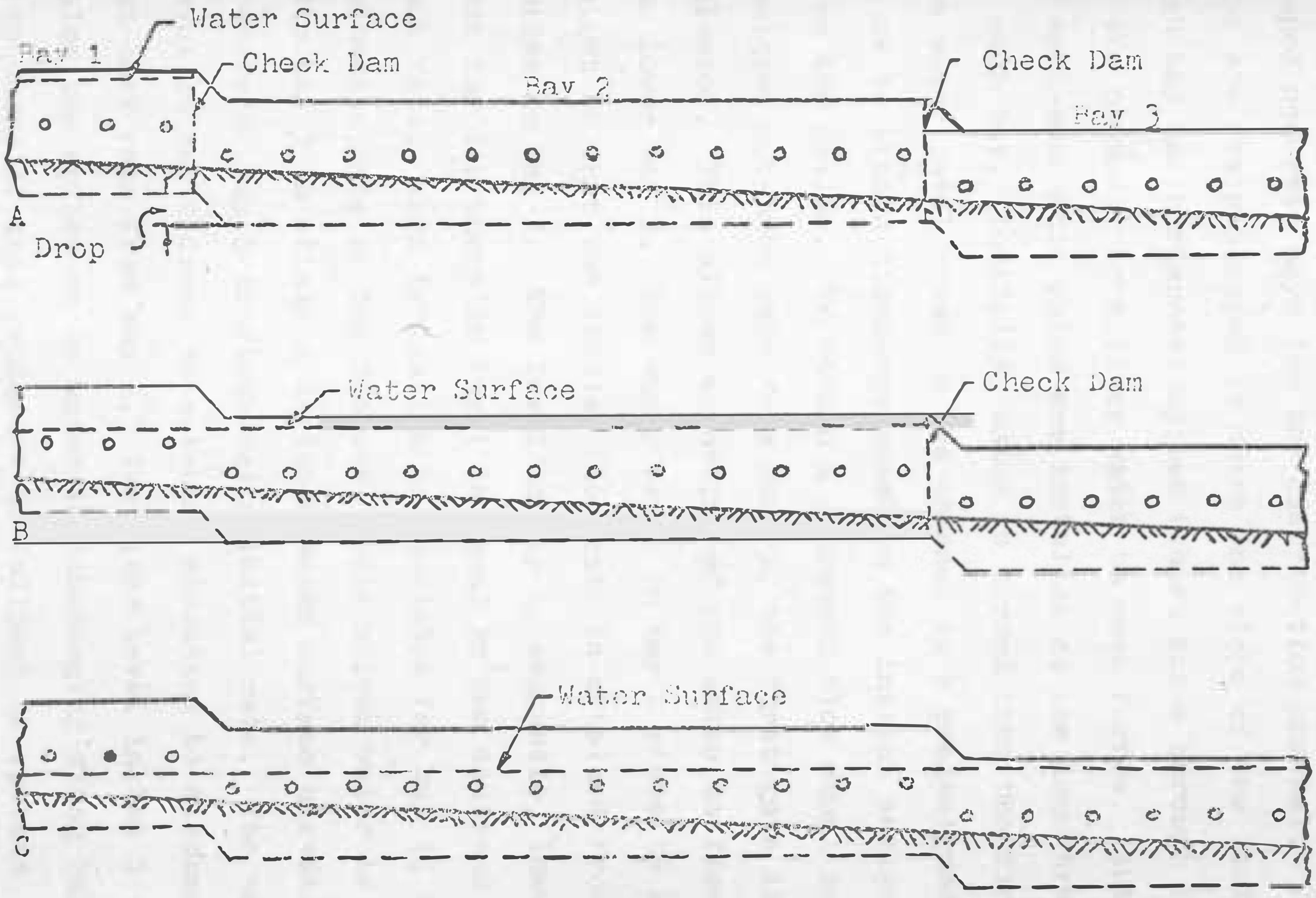

Figure 1. Drawing of Open Channel Cut-Back Furrow Irrigation System

shaped concrete bays for the distribution channel. The bays are stair-stepped to match the slope of the land. Each bay has horizontal spiles (short tubes through the side of the channel) to deliver water to each furrow. Automatic or semi-automatic gates are installed at the downstream end of each bay. Initially, water is turned into the first bay. The water level rises in the channel to a predesigned depth so as to attain discharge equal to the initial design rate from the spiles. To obtain a decreased flow equal to the designed cut-back rate from bank 1, the first gate is released. This allows a portion of the water to flow into the lower bay 2. The water level in bay 2 rises to a height so that the initial flow rate is attained from spiles in bay 2. The level in bay 1, meanwhile, lowers so that the discharge in bay 1 is equal to the designed cut-back rate. When irrigation is completed for bay 1, the automatic gate on bay 2 opens. This allows water to flow into bay 3 to attain a designed water surface elevation; therefore, bay 3 is discharging initial rate. The water level in bay 2 drops to cut-back elevation to discharge cut-back rate from bay 2. The water level in bay 1 is below the spiles so no water is discharging along bay 1. A sequence of this procedure is followed to operate remaining bays until the end of the field is reached. Garton was able to obtain a discharge variation of less than 5 percent from the spiles from each bay and between bays for both the initial and cut-back design rates. Fluctuations of the design inflow caused an increase in the discharge variation.

Uhl (26) designed a semi-portable sheet metal flume with nonregulating bored outlets for the distribution portion for Garton's cut-back procedure. The flume is constructed in 10-foot sections, so the length can be changed for different cropping practices. Uhl concludes that most of the variation of discharge occurs in the bay discharging cut-back flow. An average discharge variation of 21.5 percent for the cut-back bay was reported by Uhl for a wide range of inflows. The average percent variation for the initial bay was 3.1 percent for the same inflow range.

Paine has designed an automatic gate for installation on gated pipe. The gate, operated by water pressure, opens and closes automatically when given a signal controlled by electrical means. The gate opening is adjustable to allow a variation of flow rates. The gate is produced commercially by Toro Manufacturing Company (20).

The real evaluation of these designed systems will be determined by the installation, acceptance, and use by the surface irrigator. To truly compare and evaluate irrigation systems, a common reference and measurement should be used.

Two references that are commonly used to relate the capability of a practical system to that of an ideal irrigation system are water application efficiency and uniformity of application.

The first term, irrigation efficiency (water application efficiency), is the ratio of the volume of irrigation water held in the root zone to the total volume applied. This is generally expressed as a percentage making the equation of the form (18):

$$Ea = 100\ W_s/W_d \qquad \text{(Equation 1)}$$

where Ea = water application efficiency,

W_s = water stored in the root zone during irrigation,

and W_d = water delivered to the field.

Uniformity of application expresses the uniformity of water storage in the root zone along the field length. For research purposes, it is referred to as a water uniformity coefficient, the equation form being (18):

$$U_c = 100\ (1-(y/d)) \qquad \text{(Equation 2)}$$

where U_c = water uniformity coefficient,

y = average numerical deviation in depth of water stored from the average depth of water stored,

and d = average depth of water stored during irrigation.

After an understanding of what has been and what is being done to improve irrigation methods and techniques has been gained, it is then possible to develop new systems, methods, and techniques with desirable advantages. Such is the case for the proposal of the automated closed conduit cut-back furrow irrigation system.

THE PROPOSED SYSTEM

Cut-back irrigation requires two flow rates, a large initial flow rate to wet the furrow length, followed by a smaller cut-back rate that matches the intake rate of the soil along the furrow. Automation of the cut-back procedure using nonregulating outlets along the distribution bank required pressure differential rather than outlet size adjustment to change discharge rates. Garton (10) and Uhl (26) control the water elevation in their systems above the outlet for desired variation of discharge. For a closed pipe distribution bank with nonregulating outlets, a desired discharge rate could be obtained by pressure head control. The greater the pressure inside the pipe, the greater the outlet discharge.

Automation of the cut-back procedure in the closed pipe system requires three components: a mainline, distribution banks, and automatic pressure regulating valves. The mainline supplies water to the distribution banks. An automatic pressure regulating valve between the mainline and a distribution bank controls the pressure inside the bank for discharge of both initial and cut-back flow rates.

The Distribution Banks

The distribution banks constructed of portable, quick coupling aluminum pipe are adaptable for different cropping practices. Outlets made by boring the proper size circular holes along the bank to match the furrow spacings would give the control for discharge of initial and cut-back flow rates. A drawing of the proposed distribution bank is shown in Figure 2.

The valve and tee placement in the center of the bank forms two symmetrical sections. This allows the design length of the bank and design inflow rate to be one-half the total bank length and one-half the total inflow rate.

The uniformity of discharge from the outlets spaced along the distribution bank is necessary for even water distribution along the field width. The well designed distribution bank should have an uniformity of discharge from outlets of $\pm$ 5.0 percent of the designed initial and cut-back flow rates. This variation of discharge is used as the design limit.

Automatic Pressure Regulating Valve

The required valve is designed to respond to three different signals. On signal one, the valve will open, adjust to and maintain a constant head designed to deliver initial flow rate; on signal two, the valve will adjust to

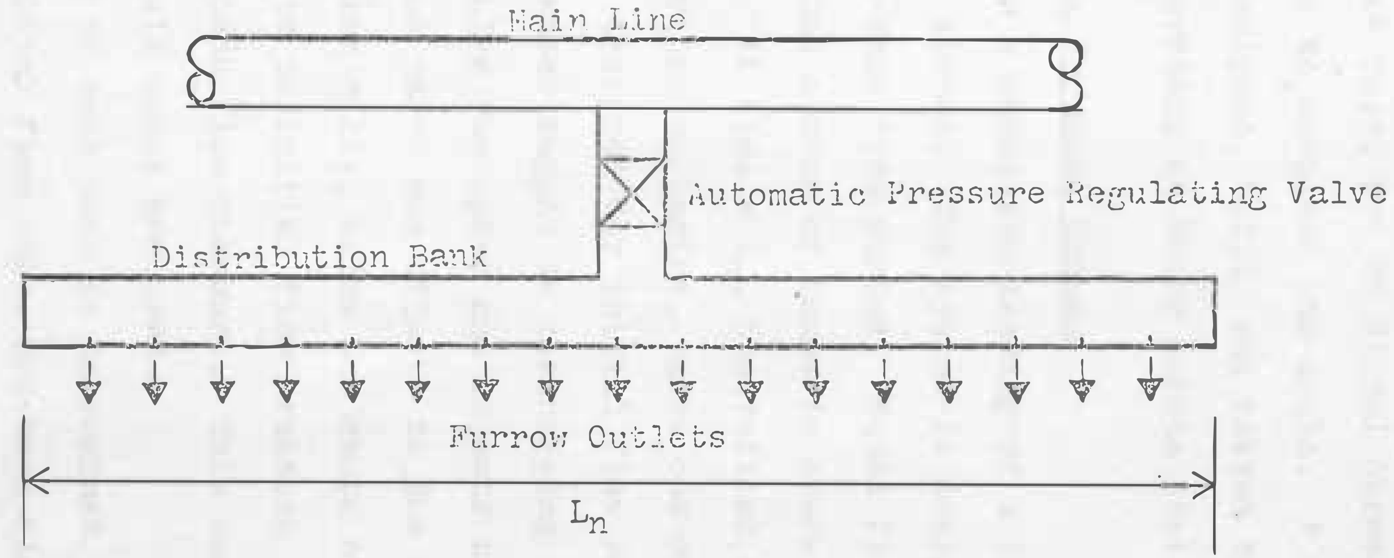

Figure 2. Drawing of the Proposed Distribution Bank

and maintain a lower constant head which will deliver the cut-back flow rate; and on signal three, the valve will completely close to complete the cycle. A valve of this type is being designed, built, and tested by the Electrical Engineering Department at South Dakota State University.

Operation of the Proposed System

Figure 3 is a schematic drawing of a field installation of the proposed system. The system is designed for an equal initial and cut-back flow period. Equal flow periods reduce the maximum number of banks in operation simultaneously to two. At time = t_o, the initial starting time of a cycle when irrigation begins, valve one opens and adjusts to the pressure designed for initial flow rate. At this time the total water supply is discharging from bank one. At time = t_1, valve two opens and adjusts to the initial flow pressure and valve one adjusts to the cut-back flow pressure. At time = $2t_1$, valve one shuts off, valve three opens and adjusts to initial flow pressure while valve two adjusts to cut-back flow pressure. This sequence is followed until all banks are used.

The length of each bank is determined by the water supply rate, initial flow rate, cut-back flow rate, and the

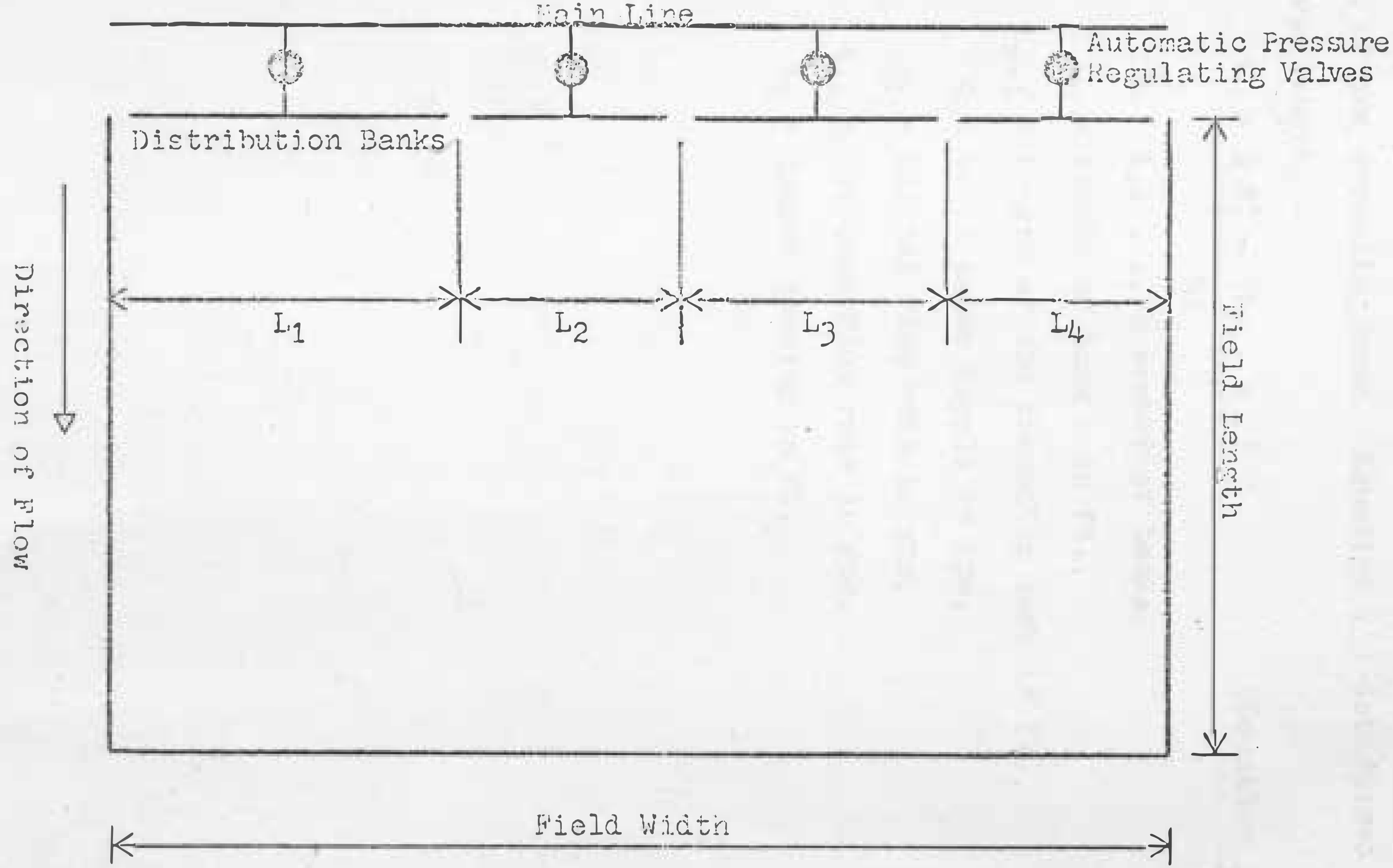

Figure 3. Schematic of Field Installation of the Proposed System

length of the preceding bank. Equation (3) determines the bank length:

$$L_n = \frac{Q\ S_p - (L_{n-1}\ q_{cb})}{q_i} \qquad \text{(Equation 3)}$$

where $n = 1, \ldots, n$ number of banks,

L_n = length of bank n in ft.,

L_{n-1} = length of the preceding bank in ft.,

Q = total water supply in gpm,

q_i = initial flow rate in gpm,

q_{cb} = cut-back flow rate in gpm,

and S_p = outlet spacing in ft.

THEORETICAL ASPECTS

The flow of water in the surface irrigation system is generally classified into two regimes, open channel and closed pipe flow. The two types of flow differ in that open channel flow must have a free water surface which is subject to atmospheric pressure, whereas full pipe flow is under a hydraulic pressure.

Closed Pipe Flow

The total energy for closed pipe flow with reference to a datum line is the sum of hydraulic pressure, H; the elevation difference between datum line and the center of the pipe, Z; the velocity head, $V^2/2g$; and the loss of energy due to friction represented by H_L. Figure 4 is a graphic representation of the energy in closed pipe flow. The universally used Bernoulli's equation represents the energy for closed pipe flow in the form:

$$H_1 + Z_1 + \frac{V_1^2}{2g} = H_2 + Z_2 + \frac{V_2^2}{2g} + H_L \quad \text{(Equation 4)}$$

where H = hydraulic or static head,

Z = elevation from datum line,

V = mean velocity inside the pipe,

g = gravitational constant,

and H_L = loss of energy due to friction.

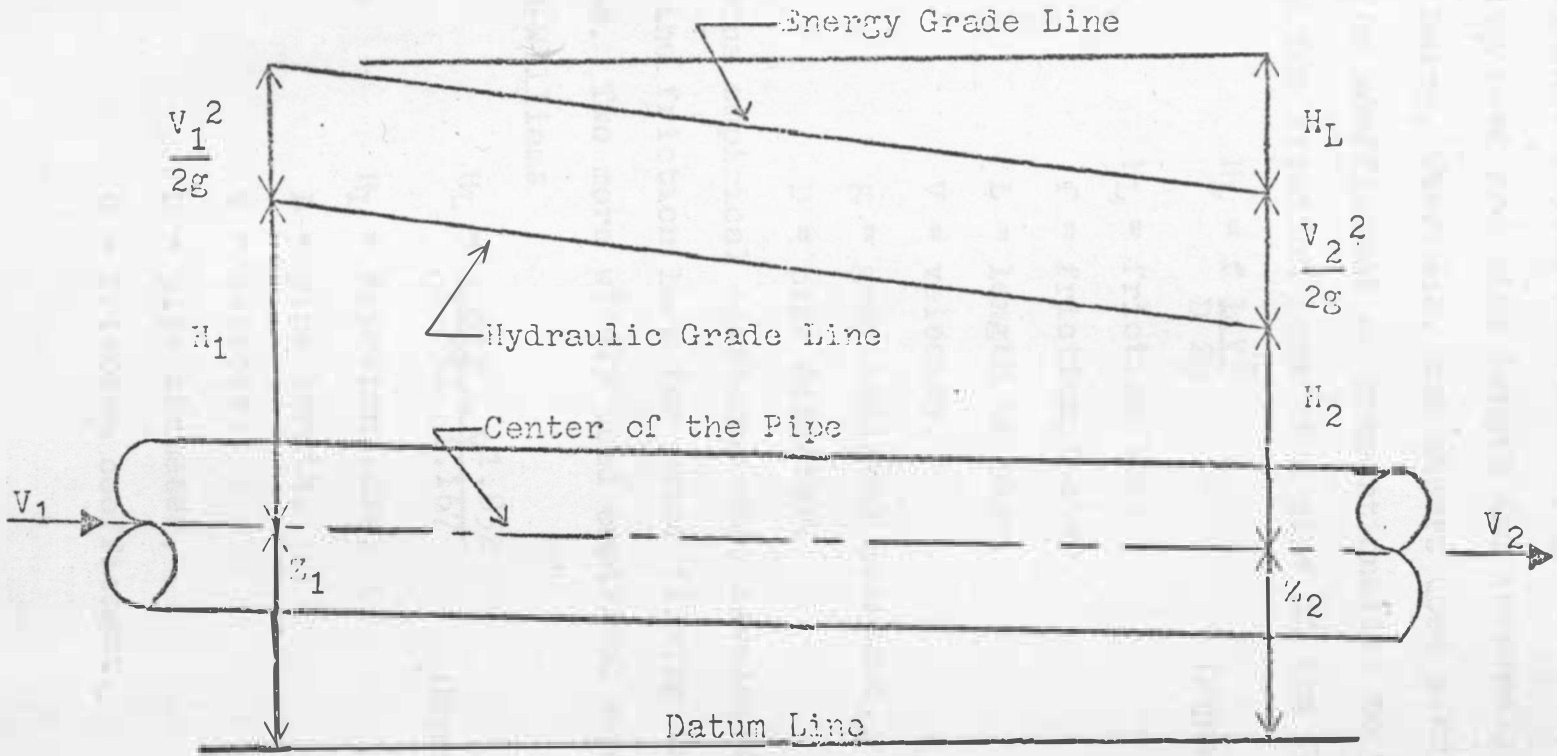

Figure 4. Graphic Representation of Total Energy for Closed Pipe Flow

Early experiments showed head loss (H_L) varied directly with velocity head and pipe length and inversely with pipe diameter. Darcy, Wiesbach, and others used a friction factor (f) or coefficient of proportionality to propose an equation for friction loss of a pipe of the form:

$$H_L = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{(Equation 5)}$$

where H_L = friction loss,

f = friction factor,

L = length of pipe,

V = velocity,

g = gravitational constant,

and D = pipe diameter.

Numerous empirical equations were developed later to calculate the friction loss for water flowing through a closed pipe. Two more widely used empirical equations are:

Hazen-Williams

$$H_L = \frac{3.022\, L\, V^{1.852}}{C^{1.852}\, D^{1.167}} \qquad \text{(Equation 6)}$$

where H_L = friction loss,

L = pipe length,

V = velocity,

D = pipe diameter,

and C = friction coefficient.

Scobeys

$$H_L = \frac{K_S\ L\ V^{1.9}}{1000\ D^{1.1}} \qquad \text{(Equation 7)}$$

where H_L = friction loss in feet of water,

K_S = coefficient of retardation,

V = mean velocity in feet per second,

D = pipe diameter in feet,

and L = pipe length in feet.

The values for C and K_S in the preceding equations are dependent on material used for pipe construction. A value of $K_S = 0.4$ is commonly used in industrial design for aluminum pipe. Lytle and Wimberly (21) report an average K_S value for several sizes of aluminum pipe and coupler combination tested. The average values of K_S are 0.25 for 3-inch diameter pipe, 0.31 for 4-inch diameter pipe, 0.23 for 5-inch diameter pipe, and 0.26 for 6-inch diameter pipe.

Spatially Varied Flow

Spatially varied flow is a form of full pipe flow when water is either removed or added through outlets spaced along the length of the pipe. There are two types, increasing spatially varied flow and decreasing spatially varied flow. Increasing spatially varied flow occurs when water is added, for example in storm sewers and drainage tile, whereas decreasing spatially varied flow occurs when water is removed. Common examples are surface irrigation distribution lines (gated pipe) and sprinkler irrigation laterals. A graphical representation of the energy line for decreasing spatially varied flow is shown in Figure 5.

The purpose of a well designed distribution system is to discharge a uniform rate of water from outlets spaced along the pipe. Several researchers have investigated the hydraulics of spatially varied flow to evaluate discharge uniformity.

To obtain an accurate analysis of spatially varied flow in an irrigation distribution bank, it is necessary to start at the last outlet and work back to the beginning, treating each section between two outlets as full pipe flow. Attempts to overcome this time-consuming and tedious procedure were developed to analyze friction loss in the distribution bank in terms of the entire lateral length. Christianson (2) derived a F factor to determine frictional loss for multiple outlet pipe in terms of frictional loss of a mainline pipe. The head loss in multiple outlet pipe is represented by the equation:

$$H_L = F \frac{(K\ L\ Q^m)}{D^{2m+n}} \qquad \text{(Equation 8)}$$

where H_L = friction loss,

F = friction factor,

K = friction coefficient,

L = length of multiple outlet pipe,

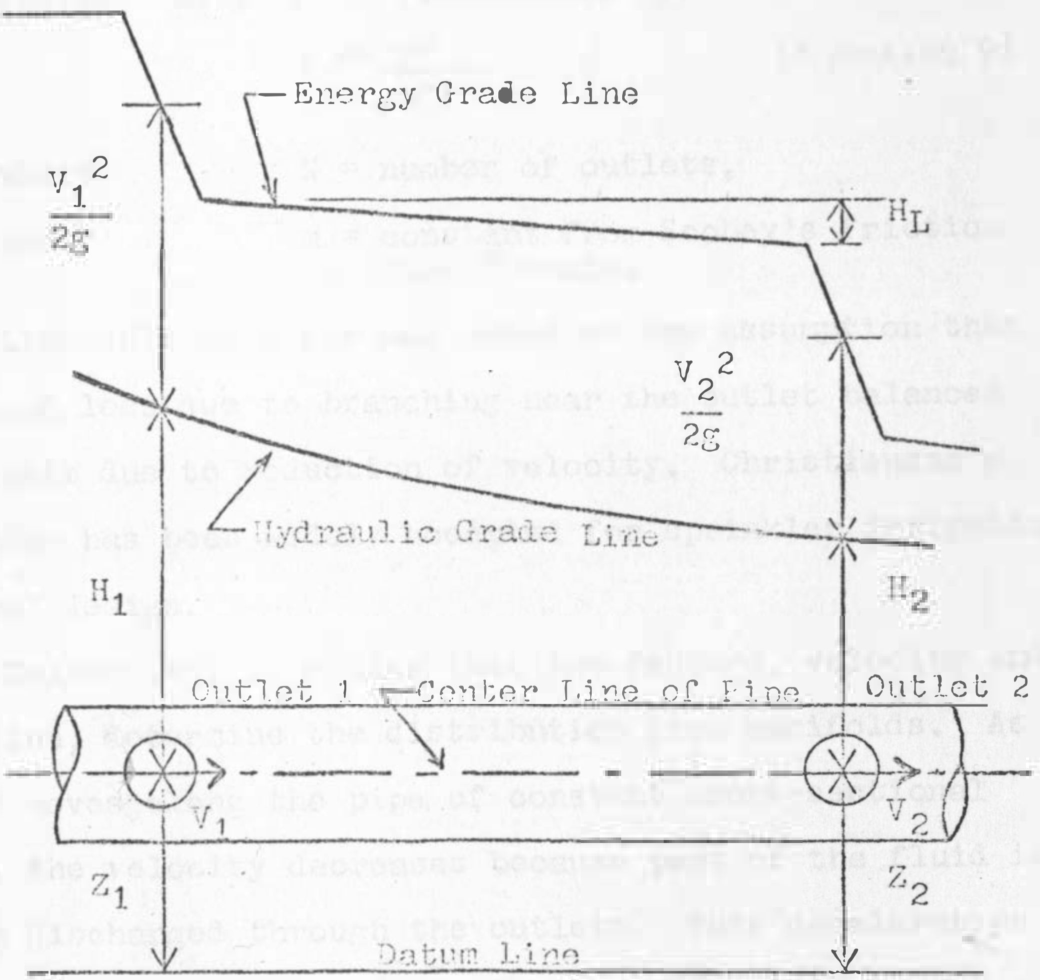

Figure 5. Graphic Illustration of the Energy Line for Decreasing Spatially Varied Flow

Q = total flow rate,

D = diameter of pipe,

and m,n = constants from Scobey's friction loss equation.

The friction factor F is represented by:

$$F = \Sigma \frac{N^m}{N^{m+1}} \qquad \text{(Equation 9)}$$

where N = number of outlets,

and m = constant from Scobey's friction loss formula.

Christianson's analysis was based on the assumption that the head loss due to branching near the outlet balances head gain due to reduction of velocity. Christianson's F factor has been widely accepted for sprinkler irrigation lateral design.

Keller (19) concludes that two factors, velocity and friction, determine the distribution from manifolds. As fluid moves along the pipe of constant cross-sectional area, the velocity decreases because part of the fluid is being discharged through the outlets. This deceleration causes an increase in pressure whereas friction causes a decrease in pressure. Keller derived the equation for pressure rise in the direction of flow for horizontal pipe as:

$$dP = -\frac{d(V^2)}{2g} + \left(f \frac{dS}{D} \frac{V^2}{W}\right) \qquad \text{(Equation 10)}$$

where P = pressure of fluid in lbs. per ft.^{2}, W = specific weight of fluid in $ft.^3$ per lb.,

V = velocity in ft. per sec.,

f = Darcy's friction factor,

D = diameter of the pipe in ft.,

g = acceleration of gravity in ft. per $sec.^2$,

and S = distance from dead end in ft.

The sign of the first term is negative because the decrease in velocity increases pressure. The second term is positive because friction causes a decrease in pressure. dS is negative in direction of flow because distance S is measured from the dead end.

The pressure at any point P, a distance S from dead end, would equal:

$$P = P_o + \frac{V_o^2 - V^2}{2g} + \int_L^S f \frac{V^2}{D} dS \qquad \text{(Equation 11)}$$

P_o and V_o are pressure and velocity at the inlet end, respectively.

For the assumption of uniform discharge, the velocity is linear; therefore,

$$V = V_o \frac{S}{L} \qquad \text{(Equation 12)}$$

where V = velocity at any point in ft. per sec.,

and S = length to point in ft.

By combining Equations 11 and 12 and integrating, the equation for pressure at any point P becomes:

$$P = P_o + \frac{V_o^2}{2g}\left[1 - (S/L)^2\right] - f\frac{V_o^2}{3D}L\left[1 - (S/L)^3\right] \qquad \text{(Equation 13)}$$

Keller also recognized uniform discharge could be achieved by changing the size of the holes along the manifold, the spacing of the outlets, or the shape of the distribution channel.

Guzman and Manges (13) modified Keller's equation to determine a slope which caused the least variation of pressure along the lateral. The equation applies to a lateral with an infinite number of outlets spaced evenly along the pipe:

$$H = H_o - \frac{V_1^2}{2g}\left[\frac{X}{X_1}\right]^2 + \left[\frac{V_1}{1.318C_H}\right]^{1.85} \frac{X_1}{2.85R^{1.117}}\left[\frac{X}{X_1}\right]^{2.85} + \frac{SXX_1}{X_1} \qquad \text{(Equation 14)}$$

where

H = pressure head at a point within the lateral,

H_o = pressure head at the dead end of the lateral,

V_1 = velocity at the open end of the lateral,

X = the distance from the point to dead end,

X_1 = the length of the lateral,

C_H = Hazen-Williams friction coefficient,

R = hydraulic radius of the pipe,

and S = slope of the lateral.

The equation was used to evaluate the head along the lateral as a function of X/X_1 for various values of S for given values of V, C_H, X, and R. The slope which gave the least variation in head over any of $0<X/X_1\leq 1$ was selected as best grade. Assuming a constant orifice coefficient along the pipe, the variation in discharge along the pipe was 2.5 percent.

Chu and Moe (3) concluded that by letting the slope of the lateral match the theoretical hydraulic gradient of the lateral, uniform discharge could be achieved from all outlets. Due to the fact that the lateral slope cannot be changed from outlet to outlet, Chu and Moe's procedure develops an average slope for each section. Chu and Moe derived the following equation to express elevation difference required along the lateral for uniform discharge:

$$Z_N - Z_1 = 1/2\, F_1^2\, L \left[\frac{K}{3} N^3 - \left(1+\frac{K}{2}\right) N^2 + \left(1+\frac{5K}{12}\right) N - \frac{K}{4} \right] + S_{fL}\, L \sum_{1}^{N-1} i\, m \quad \text{(Equation 15)}$$

where N = the total number of outlets along the lateral,

Z_N = the elevation of the Nth outlet,

Z_1 = the elevation of the first outlet at dead end,

F_1 = the Froude number (with respect to length) of the first section of the lateral from dead end,

$1/2\ F_1{}^2 = (q/A)^2 / 2gL,$

q = individual outlet discharge,

A = cross-sectional area of the lateral pipe,

g = gravitational constant,

L = spacing of outlets,

$S_{f1} = C_q m/D^{2m+n}$ = the slope of the energy grade line of the first section of the lateral counting from dead end,

$C_q m/D^{2m+n}$ = Scobey friction equation,

and K = coefficient of energy loss of head.

When the lateral is horizontal, and uniform discharge is assumed, the equation expressed pressure differential between outlets.

Outlet Discharge

The circular orifice has many uses in engineering practice. The most common of these is fluid measurement. Torricelli's equation expressed the flow from a circular orifice discharging into the atmosphere as:

$$Q = CA\ (2gh)^{\frac{1}{2}} \qquad \text{(Equation 16)}$$

where Q = orifice discharge in ft.^{3} per sec.,

A = area of the orifice in ft.^{2},

g = gravitational constant in ft. per sec.^{2},

h = static head in ft.,

and C = orifice discharge coefficient.

The C value is dependent on the shape and the method of construction of the orifice. Values of C range from 0.61 for sharp-edged orifices to 0.98 for rounded orifices (28).

Greve (12) investigated the flow from vertical, circular orifices. The diameter of orifices studied ranged from 0.25 to 2.495 feet. Greve used an empirical equation of the form $Q = M(H)^N$ to represent his results.

Fry (8) concluded that the discharge from gates on gated pipe is dependent on the following factors:

1. the available pressure head at each gate
2. the velocity inside the pipe past the gate
3. the fractional amount of gate opening.

Fry reported that a velocity increase from 0.0 to 4.0 feet per second past the gate reduced the discharge as much as 25 percent when gate size and head were held constant. At higher heads, a velocity increase caused less variation on discharge rates.

Guzman and Manges (13) reported Toricilli's equation overestimated discharge from outlets at inlet end while underestimating discharge from outlets at dead end. They concluded that this indicated the orifice coefficient varied inversely with distance from the dead end of the lateral.

LABORATORY APPARATUS, MEASUREMENTS, AND PROCEDURE

In order to achieve the objectives of this study, the laboratory apparatus was constructed to evaluate the following:

1. Discharge characteristics for nonregulating bored outlets as a function of (a) static head and (b) a combination of static head and velocity past the outlet

2. Discharge uniformity from outlets along a distribution section as a function of pressure variation along the section.

The Hydraulics Laboratory located in the Agricultural Engineering Building on the campus of South Dakota State University, Brookings, South Dakota, was used to house the experimental apparatus. The apparatus consisted of adapting the existing facilities to a test section and the test section constructed by the author. Figure 6 shows a top view of the complete apparatus used.

The test section, shown in Figure 7, was constructed from a section of 8-inch portable aluminum irrigation pipe 30 feet in length with a thickness of 0.05 inches. Initially, 10 outlets, 0.75-inch in diameter, were placed horizontally along the section of pipe at 3.0-foot intervals. The center of the first outlet was located 1.5 feet from the upstream end of the section. Initial outlets were constructed with an electric drill and 0.75-inch bit. The initial outlets were enlarged for the testing of the 1.00- and 1.25-inch outlets.

Directly opposite each outlet, a 0.25-inch hole was drilled for manometer tap placement. The manometer taps were made by drilling a 0.0625-inch hole through a 0.25 inch steel bolt 1.0 inch long. The bolts were placed in the 0.25-inch holes with the head on the inside of the pipe. Rubber gaskets and nuts were placed on the threads on the outside of the pipe to prevent leakage and to secure the manometer taps. An additional manometer tap was placed 1.5 feet upstream from the test section in the delivery pipe. Outlets and manometers were designated as shown in Figure 6.

Testing Procedure

A series of tests was made for each of the three outlet sizes. Each series consisted of tests with 6.0, 12.0, 24.0, and 36.0 inches of static head at manometer 0. The initial test at each pressure was made for no outflow out the end of the test section. This allowed for a minimum velocity of water inside the pipe past the outlets. Further tests at each pressure were made by adjusting inflow and outflow at the end of the test section so that measurements could be made for greater velocity ranges past the outlets. The velocity ranges tested were from 0.0 to 2.0 feet per second for heads of 6.0 and 12.0 inches and from 0.0 to 4.0 feet per second for 24.0- and 36.0-inch heads.

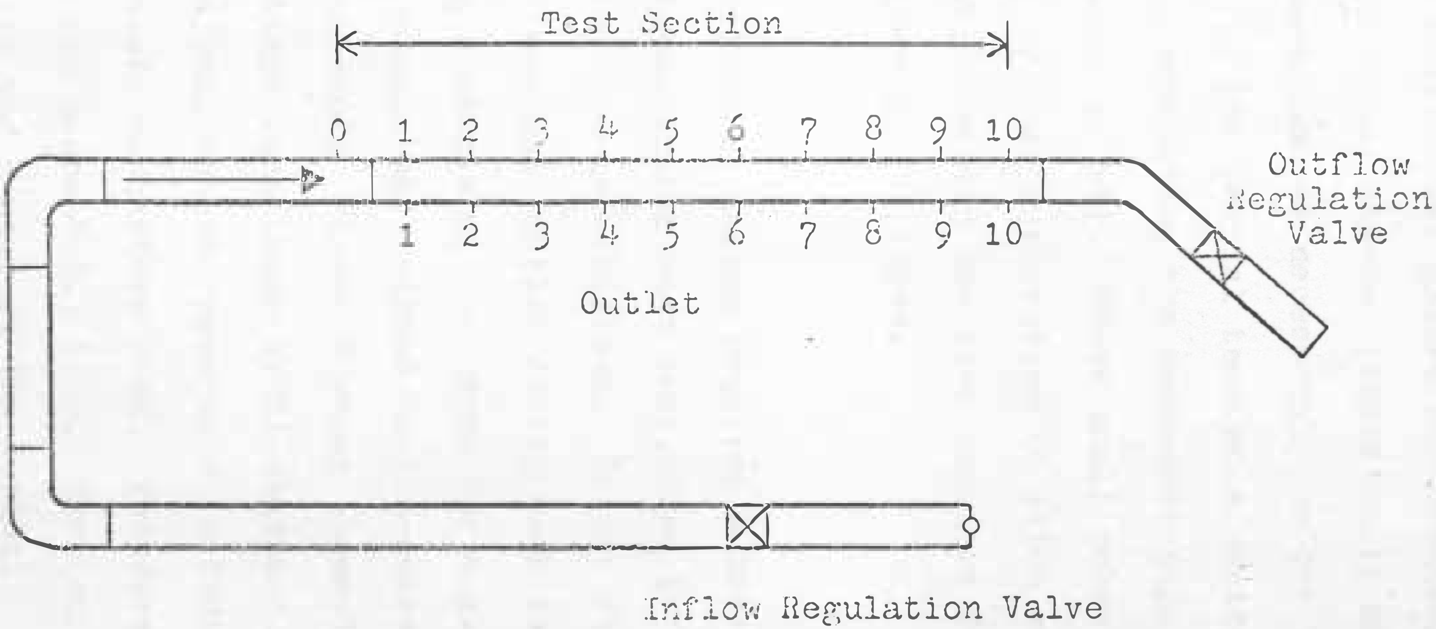

Figure 6. Drawing of Top View of Laboratory Apparatus

The test section was placed horizontally for all the tests. The level was checked periodically and minor adjustments were made as necessary. Before the pressure head was set for the initial test of a series, the plastic tubing connecting the manometer taps to the manometer board was cleared of air. Three small holes were drilled along the top of the test section to allow entrapped air to escape. This reduced the time required for flow in the test section to stabilize.

Measurements

The experimental design required a series of measurements to be taken during each test. The information was obtained in the manner explained. Outlet discharge measurements from each outlet during each test were made by collecting a sample of discharge for a given time period. The sample was weighed to the nearest pound and the weight recorded. Figure 8 shows a sample being taken. Pressure readings were taken at 11 stations along the test section. The measurements were made in inches of water from the open-air manometers used. Pressure readings were made to the nearest 0.1 inch. Zero on the manometer corresponded to the center of the outlet.

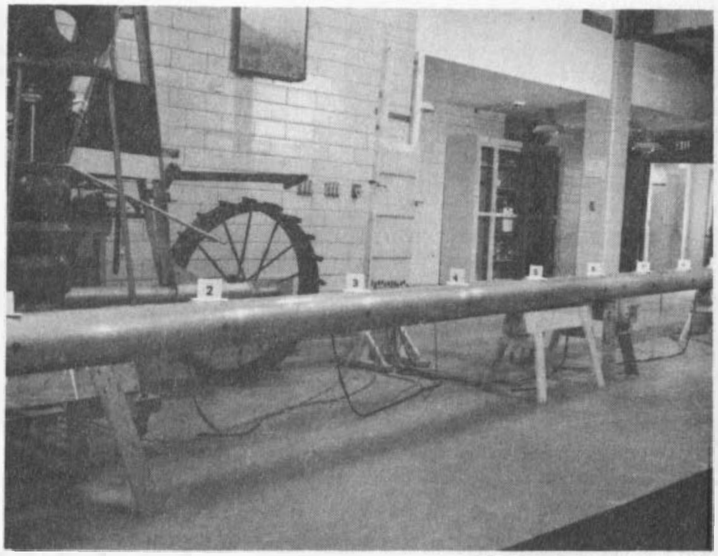

Figure 7. The Test Section

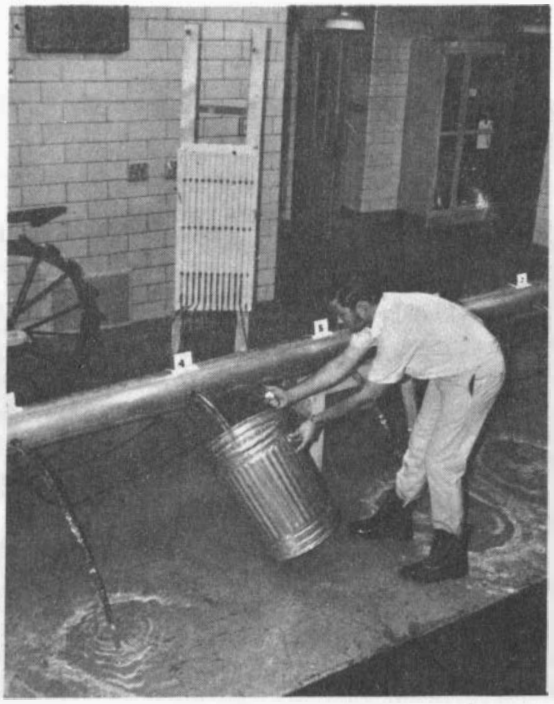

Figure 8. Measuring Outlet Discharge

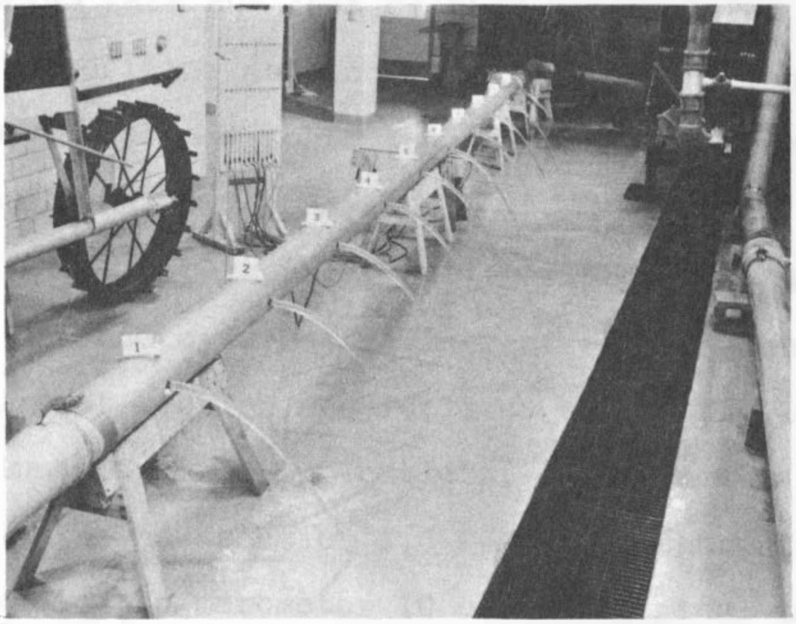

Figure 9. Test Section Discharging at 6.0 Inches of Head

Figure 10. Test Section Discharging at 36.0 Inches of Head

Inflow into the test section was measured by means of a 4.0-inch orifice and accompanying mercury manometer. Outflow was assumed to be the difference between inflow and sum of outlet discharge.

To facilitate testing for the laboratory determination of K_s (Scobey's coefficient of retardation), the original test section was replaced with a similar section of aluminum pipe with no outlets. A manometer tap was placed in the new section 1.5 feet from the outflow end corresponding to manometer 10 on the original test section. The distance between manometer 0 in the delivery pipe and the new tap was 30 feet. By regulating the inflow and outflow from the section, the head loss for 30 feet of pipe was measured. Flow rates tested were from 600 to 100 gallons per minute.

Two undesirable operating characteristics of the proposed distribution bank were noted from the operation of the test section in the laboratory. As the velocity inside the pipe past the outlet increased, the outlet discharge stream became deflected from perpendicular to the section as shown in Figure 11. Under field conditions, excessive deflection of discharge stream may cause the stream to erode the ridges between the furrows. The second characteristic was the erosion potential of the

Figure 11. Deflection of Discharge Stream

discharging stream at high heads. Evaluation as to the effect of these characteristics was left to field testing.

ANALYSIS AND DISCUSSION OF RESULTS

For each laboratory test, the hydraulic pressure and the discharge were measured at each outlet. The inflow to and outflow from the test section were also recorded. With the data from these tests, and the combination of tests, the analysis of results was made. Data form individual laboratory tests are shown in Appendix A.

Outlet Discharge Characteristics

Three factors were assumed to influence the discharge characteristics of the nonregulating bored outlets. These factors were the size of the outlet, the hydraulic pressure in the section, and the velocity of the water inside the pipe past the outlet.

Three different sizes of outlets (0.75, 1.00, and 1.25 inches in diameter) were tested. The dimension of a given outlet did not vary between tests because the size was a construction feature of the test section.

The hydraulic head factor and its relationship to discharge was analyzed first. For the decreasing spatially varied flow tests with no outflow from the end of the test section, the average velocity inside the pipe would be a minimum. The discharge and hydraulic pressure data from these tests were used to develop an equation of best fit of the form:

$$Q = C\ H^{x} \qquad \text{(Equation 17)}$$

where Q = discharge from the outlet in gallons per minute,

C = outlet discharge coefficient,

H = hydraulic pressure in inches,

and x = a constant.

A linear regression analysis run in log-log space gave the following equations of best fit for the outlet sizes tested for a pressure range from 6.0 to 36.0 inches.

0.75 inch	$Q = 1.940\ H^{0.504}$
correlation coefficient	$r = 0.998$
standard error of the estimates	$s = 0.0129$
1.00 inch	$Q = 3.545\ H^{0.492}$
	$r = 0.998$
	$s = 0.02032$
1.25 inch	$Q = 6.113\ H^{0.493}$
	$r = 0.998$
	$s = 0.0207$

Figure 12 depicts hydraulic head versus outlet discharge for the three outlet sizes.

Velocity Effect on Discharge

Analysis of the combined effect on discharge of the velocity of water inside the pipe and the static head

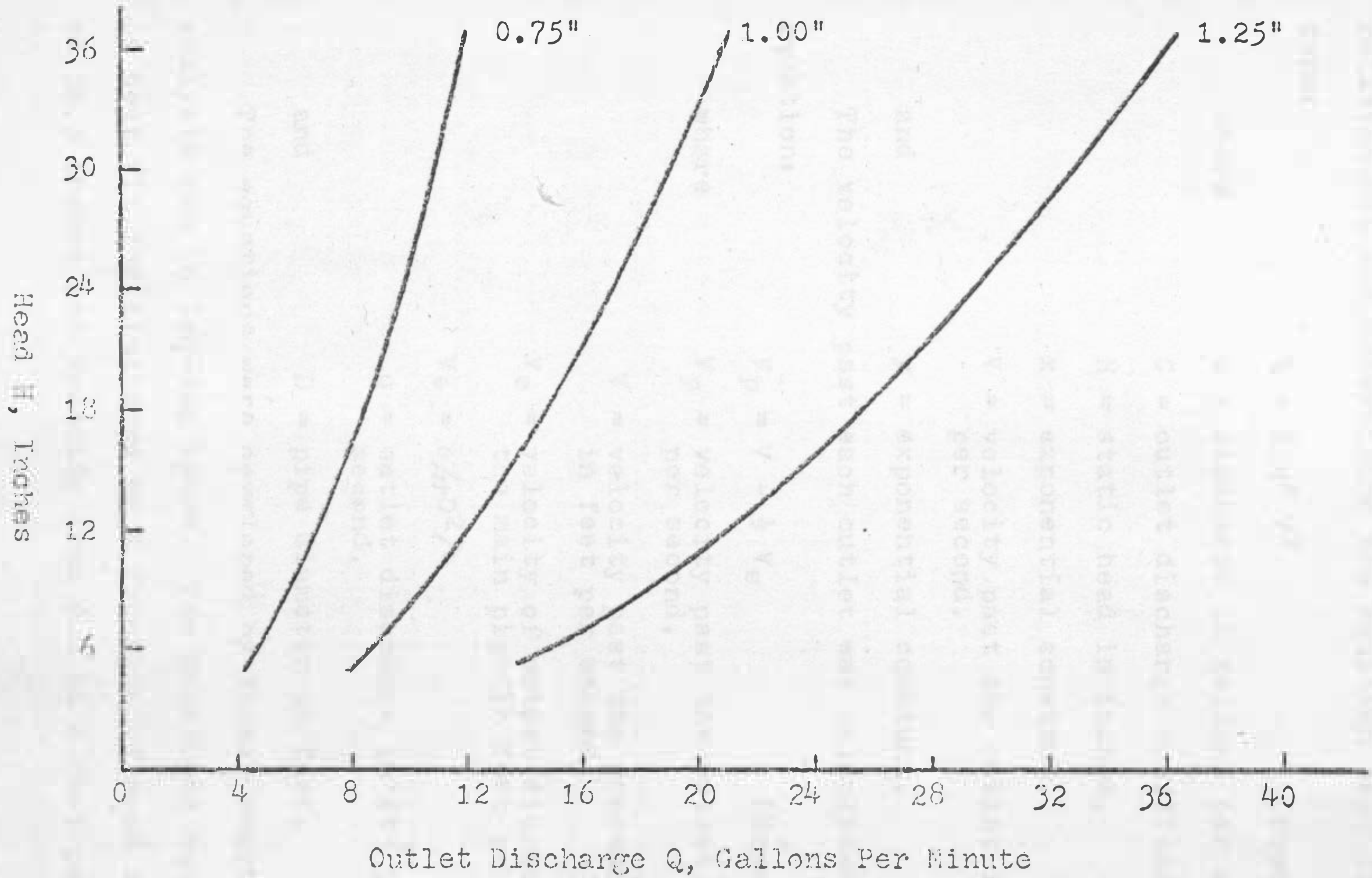

Figure 12. Head, H Versus Outlet Discharge, Q for the 0.75", 1.00", 1.25" Diameter Outlets

within the pipe was accomplished by assuming a curvilinear relationship and determining the equation best fit of the form:

$$Q = C\ H^x\ V^y \qquad \text{(Equation 18)}$$

where Q = discharge in gallons per minute,

C = outlet discharge coefficient,

H = static head in inches,

x = exponential constant,

V = velocity past the outlet in feet per second,

and y = exponential constant.

The velocity past each outlet was calculated by the equation:

$$V_p = V - \tfrac{1}{2} V_e \qquad \text{(Equation 19)}$$

where V_p = velocity past the outlet in feet per second,

V = velocity past the preceding outlet in feet per second,

V_e = velocity of outlet discharge in the main pipe in feet per second,

$V_e = q/\pi D^2/4$

q = outlet discharge in ft^3 per second,

and D = pipe diameter in feet.

The equations were developed by linear regression analysis run in log-log space. The developed equations of best fit for discharge as a function of head from 6.0 to 36.0 inches and velocity from 0.0 to 2 feet per second for head from 6.0 to 12.0 inches and 0.0 to 4.0 feet per second for a head at 24.0 and 36.0 inches are as follows:

0.75-inch outlet	$Q = 1.869\ H^{0.515}\ V^{-0.00224}$
correlation coefficient	$r = 0.998$
standard error of the estimate	$s = 0.01979$
1.00-inch outlet	$Q = 3.622\ H^{0.486}\ V^{0.00427}$
	$r = 0.998$
	$s = 0.02159$
1.25-inch outlet	$Q = 6.068\ H^{0.493}\ V^{-0.00528}$
	$r = 0.998$
	$s = 0.02191$

The outlet discharge variation for velocity ranges of 0.25 to 4.0 feet per second past an outlet at constant head are shown in Table 1. The variation for ranges shown is less than design limit set forth of $\pm$ 5% variation from the design discharge rates; therefore, velocity past the outlet can be neglected for design purposes. This makes possible the use of the Figure 12 for determining the head required for the design initial and cut-back flow rates.

Pressure Variation

The symmetrical design of the distribution bank allowed pressure variation to occur from the center each

Table 1. Velocity Factor (V^y) for the Range of Velocities Tested

	Velocity Factor		
Velocity (ft/sec)	For 0.75-In Outlet ($V^{-0.00224}$)	For 1.00-In Outlet ($V^{0.00427}$)	For 1.25-In Outlet ($V^{-0.00528}$)
0.25	1.0030	1.0059	1.0140
0.50	1.0020	1.0030	1.0037
0.75	1.0006	0.9988	1.0015
1.00	1.0000	1.0000	1.0000
1.25	0.9995	1.0009	0.9988
1.50	0.9990	1.0017	0.9979
1.75	0.9987	1.0024	0.9970
2.00	0.9984	1.0030	0.9962
3.00	0.9975	1.0047	0.9942
4.00	0.9969	1.0059	0.9927

way rather than along the entire length. Therefore, the pressure variation analysis was made for the pressure variation along one-half the total distribution bank length.

The hydraulic head was measured at 11 points along the test section for all the decreasing spatially varied flow tests made. By adjusting the inflow and outflow from the test section, the representation of a 30-foot section along the distribution bank was made possible. For example, when no outflow occurred from the outflow end of the test section, the outlets 1 to 10 on the test section as shown in Figure 6 would represent outlets 10 to 1 on a distribution bank when the last outlet on the end of the distribution bank was 1. Manometer 0 on the test section would represent the pressure of the outlet directly upstream of the 10th outlet on the distribution bank. Therefore, pressure variation for 11 points on the test section corresponds to a pressure variation from outlets 11 to 1 on the distribution bank. Likewise, for an inflow of 20 times the average outlet discharge to the test section, the test section would correspond to a section on the distribution bank between outlets 21 to 11.

The measured hydraulic pressure variation for the tests with 1.25-inch diameter outlets was compared with calculated hydraulic pressure variations for corresponding outlets along a similar distribution bank. The calculated pressure variation was computed using Equation (15). Equation (15) incorporates the use of Scobey's friction constant K_s. Experimental results show an average K_s equals 0.25 for the quick coupler 8-inch aluminum pipe used for test section. Results for experimental determination of K_s will be given later. In order to use Equation (15) to calculate pressure variation, two assumptions must be made: 1. The discharge for all outlets is constant, and 2. the coefficient of energy loss due to branching near the outlet is 0.0. The assumptions permit the obtained data to formulate an equation to determine pressure variation for a distribution bank containing up to 50 outlets. A computer program was written to solve the equation. Figure 13 is a graphical comparison of measured pressure variation for 1.25-inch outlets tests to calculated pressure variation for corresponding outlets on distribution bank. Tables 2, 3, and 4 show measured and calculated pressure variations for all spatially varied flow tests.

Graphical comparisons indicate good correlation between measured and calculated pressure variation. Therefore, Equation (15) is an acceptable tool to determine the expected pressure variation along a distribution bank for field use.

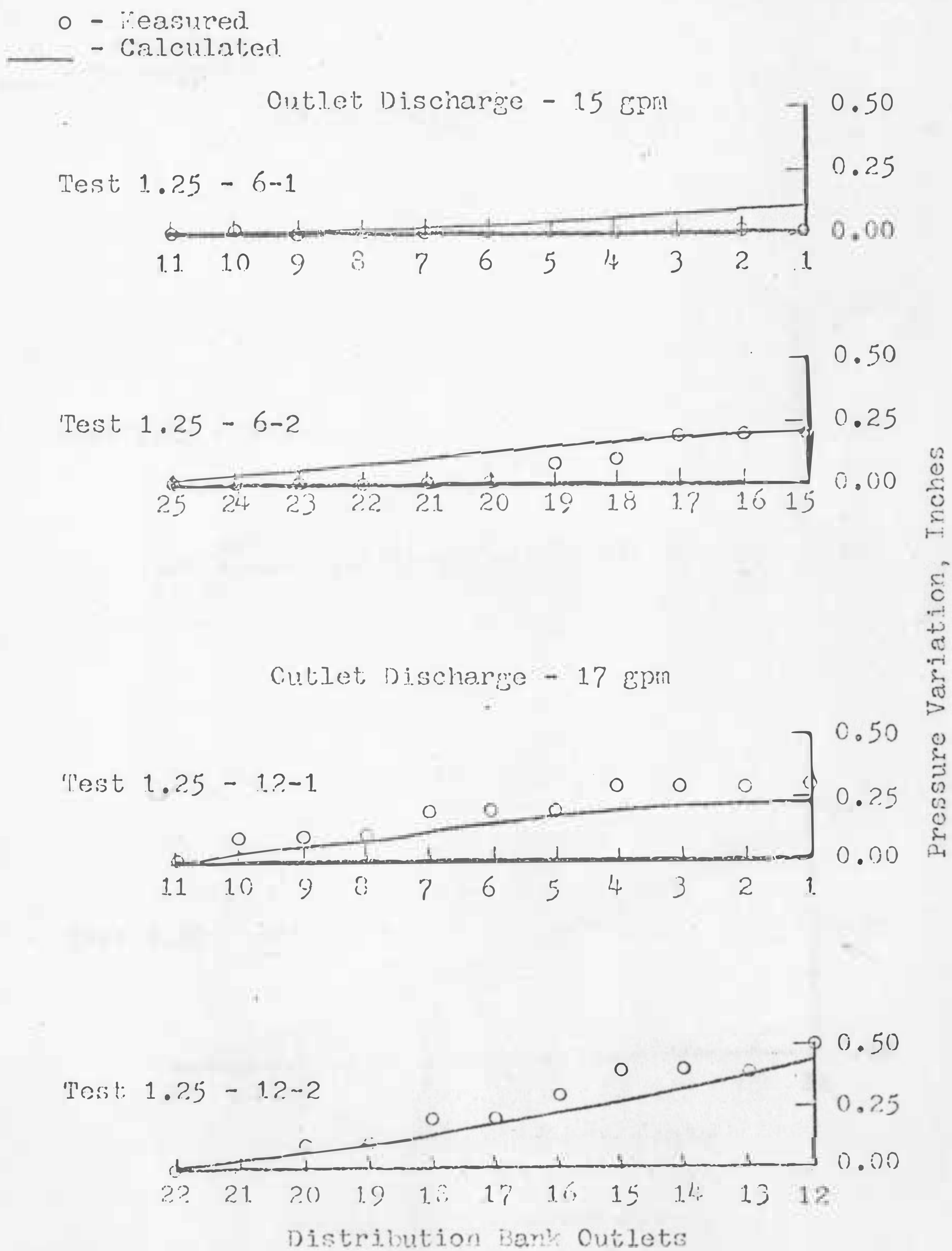

Figure 13. Measured and Calculated Pressure Variation for Test with 1.25-Inch Outlets

o - Measured
—— - Calculated

Outlet Discharge - 30 gpm

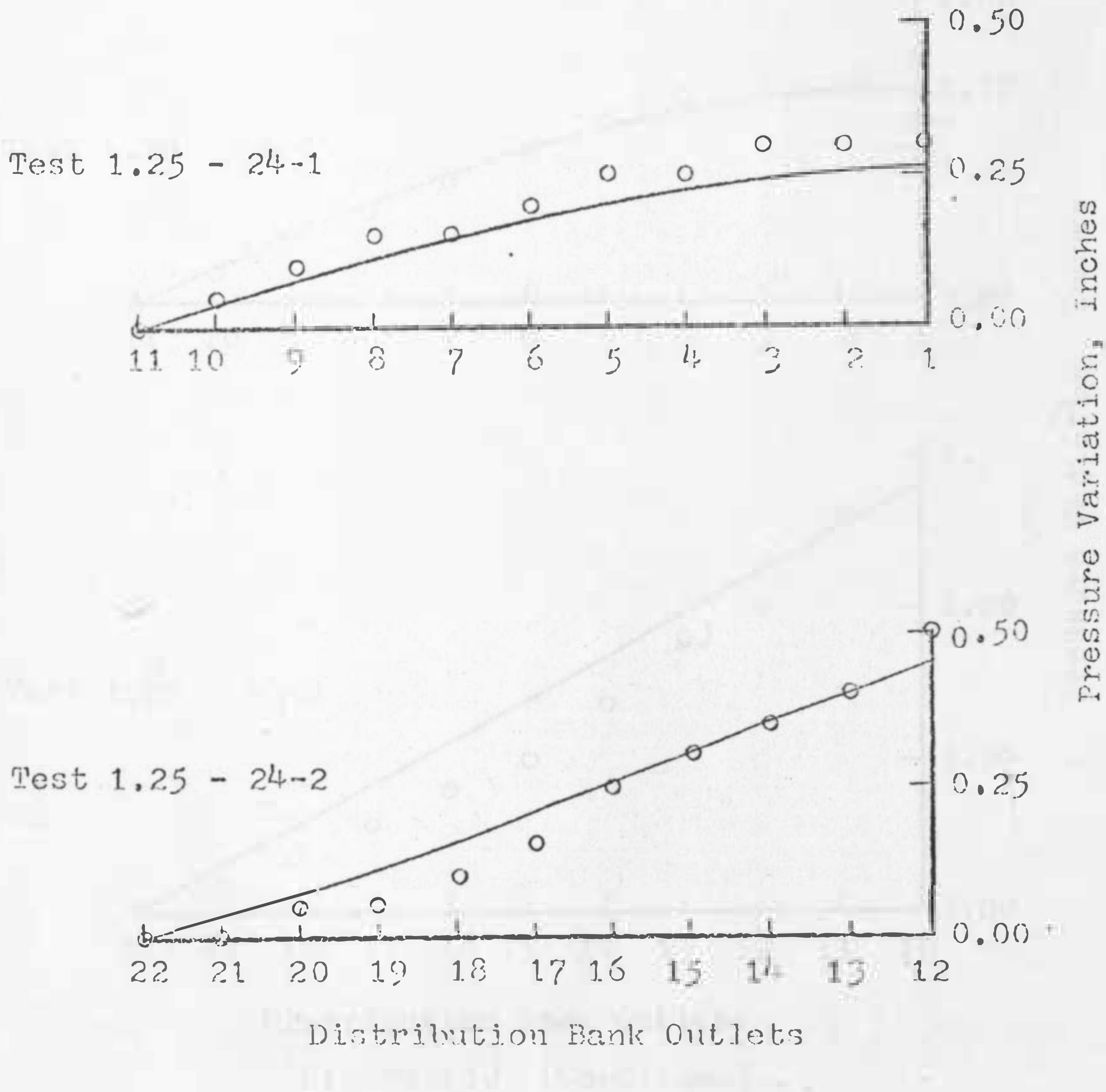

Figure 13. (Continued)

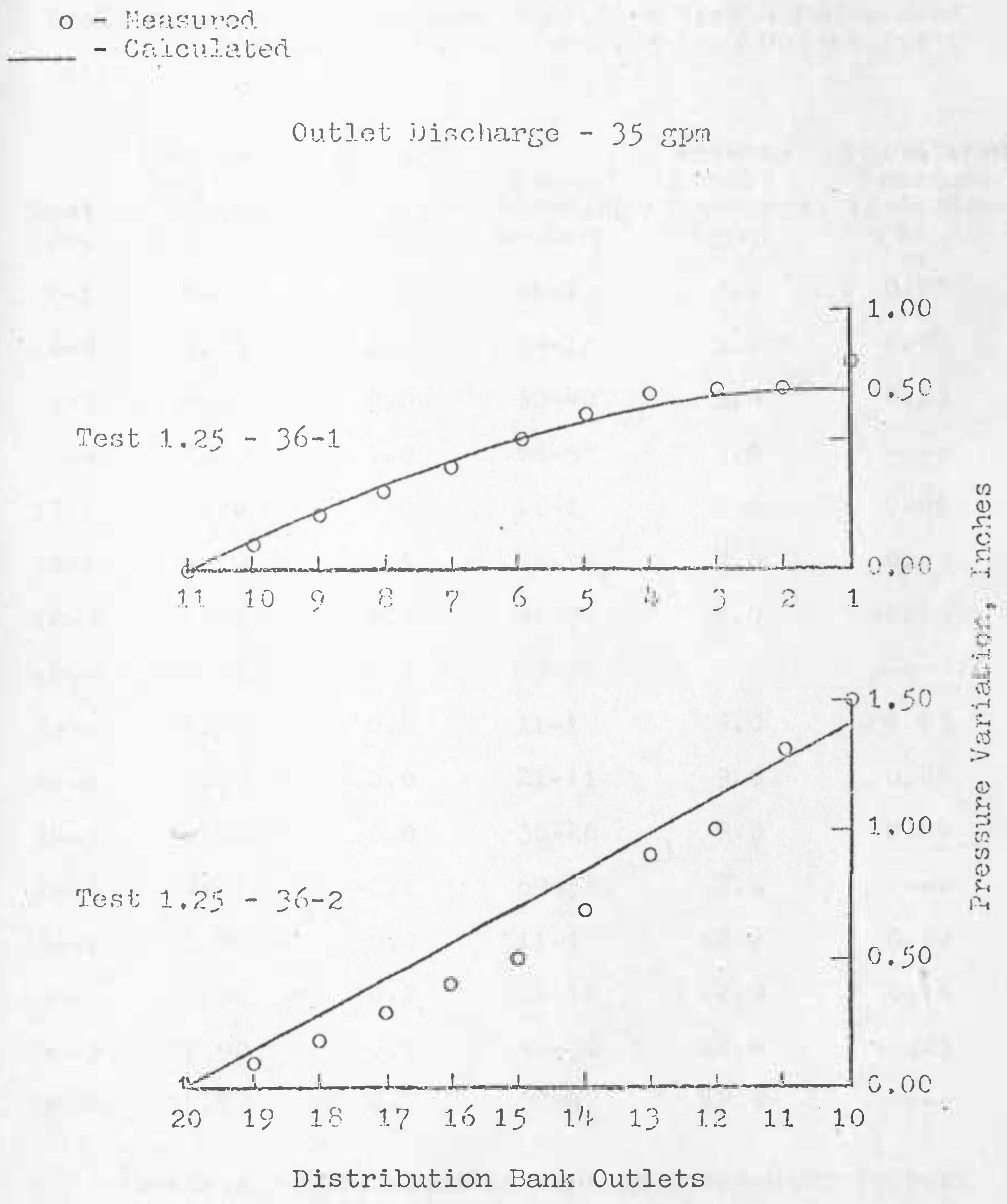

Figure 13. (Continued)

Table 2. Measured Pressure Variation Versus Calculated Pressure Variation for 0.75-Inch Outlet Tests

Test No.	Average Outlet Discharge (gpm)	Measured Pressure Variation (in)	Corre-sponding Outlets[1]	Assumed[2] Outlet Discharge (gpm)	Calculated Pressure Variation (in)
6-1	4.80	0.0	11-1	5.0	0.00
6-2	4.83	0.0	24-12	5.0	0.02
6-3	4.63	0.0	50-40	5.0	0.15
6-4	4.70	0.0	68-58	5.0	----
12-1	6.74	0.0	11-1	7.0	0.00
12-2	6.73	0.0	21-11	7.0	0.05
12-3	6.62	0.0	46-36	7.0	-0.15
12-4	6.57	-0.4	65-55	7.0	----
24-1	9.69	0.0	11-1	9.0	0.03
24-2	9.62	0.0	21-11	9.0	0.04
24-3	9.64	0.0	36-26	9.0	0.04
24-4	9.47	-0.1	69-59	9.0	----
36-1	11.70	0.0	11-1	12.0	0.09
36-2	11.98	0.2	22-12	12.0	0.14
36-3	11.90	-0.1	40-30	12.0	-0.25
36-4	11.72	-0.7	57-47	12.0	----

[1]Outlets on distribution bank corresponding to test section outlets

[2]Assumed outlet discharge for calculated pressure variation

Table 3. Measured Pressure Variation Versus Calculated Pressure Variation for 1.00-Inch Outlet Tests

Test No.	Average Outlet Discharge (gpm)	Measured Pressure Variation (in)	Corre-[1] sponding Outlets	Assumed[2] Outlet Discharge (gpm)	Calculated Pressure Variation (in)
6-1	8.53	0.0	11-1	9.0	0.04
6-2	8.63	0.0	20-10	9.0	0.04
6-3	8.75	0.0	29-19	9.0	0.01
6-4	8.61	0.0	37-27	9.0	-0.05
12-1	11.99	0.1	11-1	12.0	0.09
12-2	12.32	0.2	19-8	12.0	0.15
12-3	12.12	0.2	28-17	12.0	0.13
12-4	12.18	0.1	32-21	12.0	0.05
24-1	17.07	0.3	11-1	17.0	0.16
24-2	17.17	0.4	25-15	17.0	0.23
24-3	16.79	0.2	30-20	17.0	0.11
24-4	17.36	0.1	32-22	17.0	0.11
36-1	20.57	0.3	11-1	20.0	0.28
36-2	20.81	0.5	17-7	20.0	0.46
36-3	20.81	0.6	25-15	20.0	0.33
36-4	20.60	0.4	30-20	20.0	0.17

[1]Outlets on distribution bank corresponding to test section outlets

[2]Assumed outlet discharge for calculated pressure variation

Table 4. Measured Pressure Variation Versus Calculated Pressure Variation for 1.25-Inch Outlet Tests

Test No.	Average Outlet Discharge (gpm)	Measured Pressure Variation (in)	Corre-sponding Outlets[1]	Assumed[2] Outlet Discharge (gpm)	Calculated Pressure Variation (in)
6-1	14.77	0.0	11-1	15.0	0.12
6-2	14.65	0.2	24-14	15.0	0.18
12-1	20.96	0.3	11-1	20.0	0.27
12-2	20.81	0.5	21-11	20.0	0.40
24-1	29.74	0.6	11-1	30.0	0.80
24-2	29.11	1.0	21-11	30.0	1.00
36-1	36.02	0.8	11-1	35.0	0.71
36-2	35.25	1.5	20-10	35.0	1.30

[1]Outlets on distribution bank corresponding to test section outlets

[2]Assumed outlet discharge for calculated pressure variation

Figures 14, 15, and 16 show the calculated pressure variation expected along one-half the distribution bank with a specified number of outlets for various outlet discharge rates.

Table 5 shows the calculated discharge variation along one-half the maximum distribution bank length. The largest calculated discharge variation was 2.7 percent from design; therefore, the discharge variation caused by pressure variation is less than design limits. This allows the distribution bank to be placed horizontally for field installation.

Determination of Scobey's K_s Value

The head loss due to frictional forces is an integral part of the developed equations and figures. Since the equations used in development used the Scobey K_s value, a determination of the K_s value of the pipe used in construction of the distribution bank was made. This was accomplished by the use of Equation (7).

$$H_L = \frac{K_s \, L \, V^{1.9}}{1000 \, D^{1.1}}$$

The data collected from the closed pipe laboratory tests were used to solve for K_s for a flow range of 100 to 600 gallons per minute. Calculated K_s values are shown in Table 6.

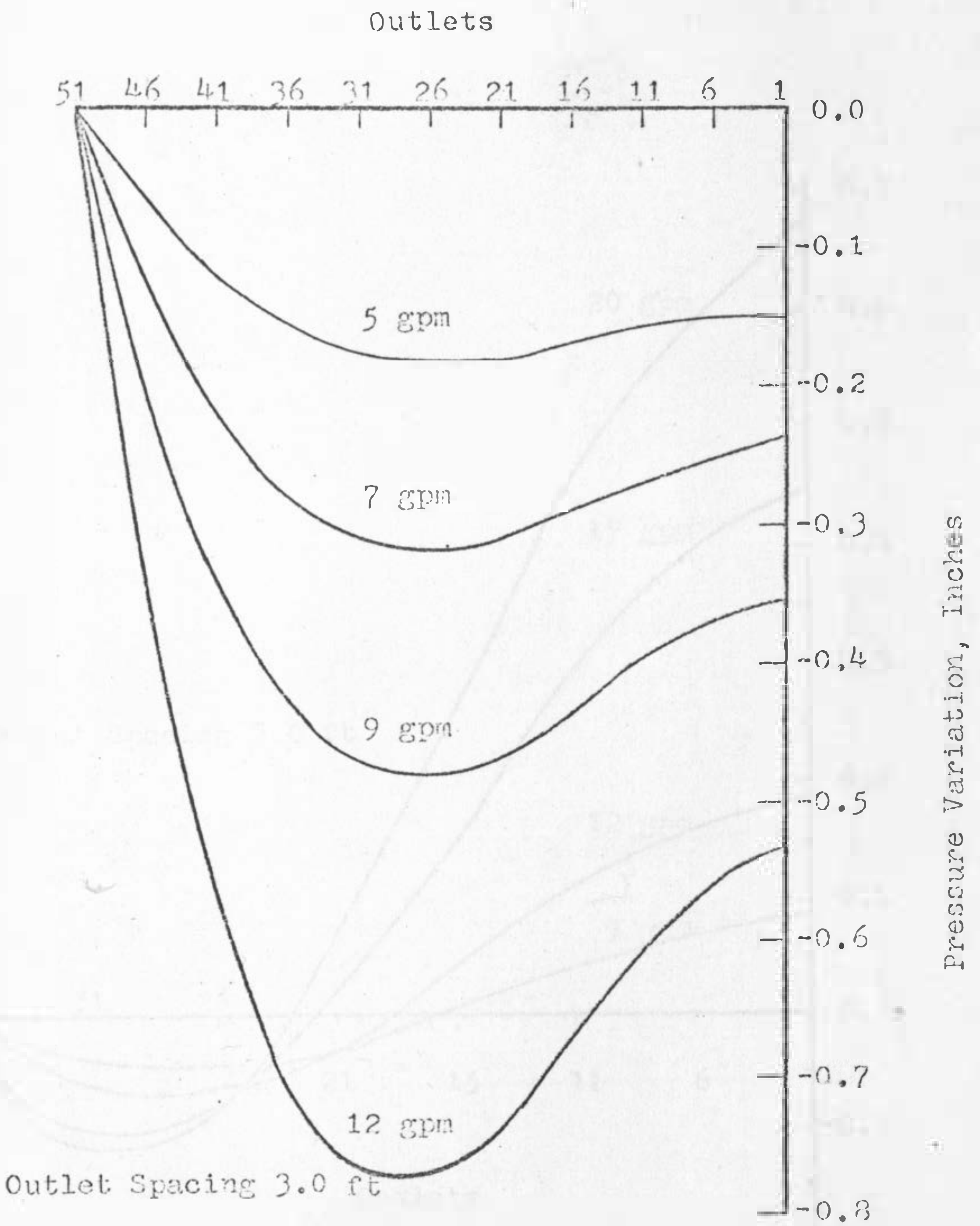

Figure 14. Calculated Pressure Variation for One-Half the Distribution Bank with 0.75-inch Outlet for 5, 7, 9, and 12 Gallons Per Minute Outlet Discharge

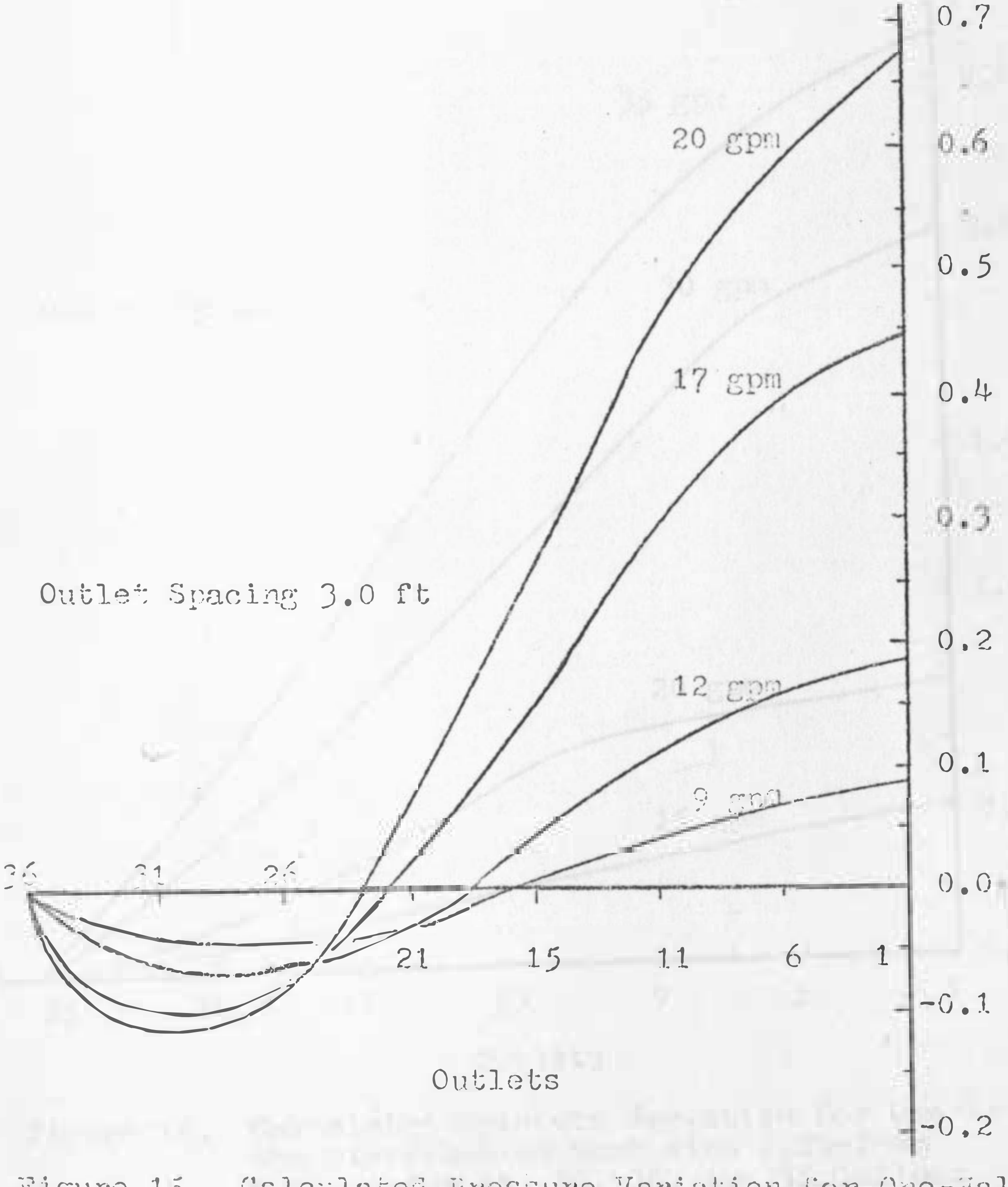

Figure 15. Calculated Pressure Variation for One-Half the Distribution Bank with 1.00-Inch Outlets for 9, 12, 17, and 20 Gallons Per Minute Outlet Discharge

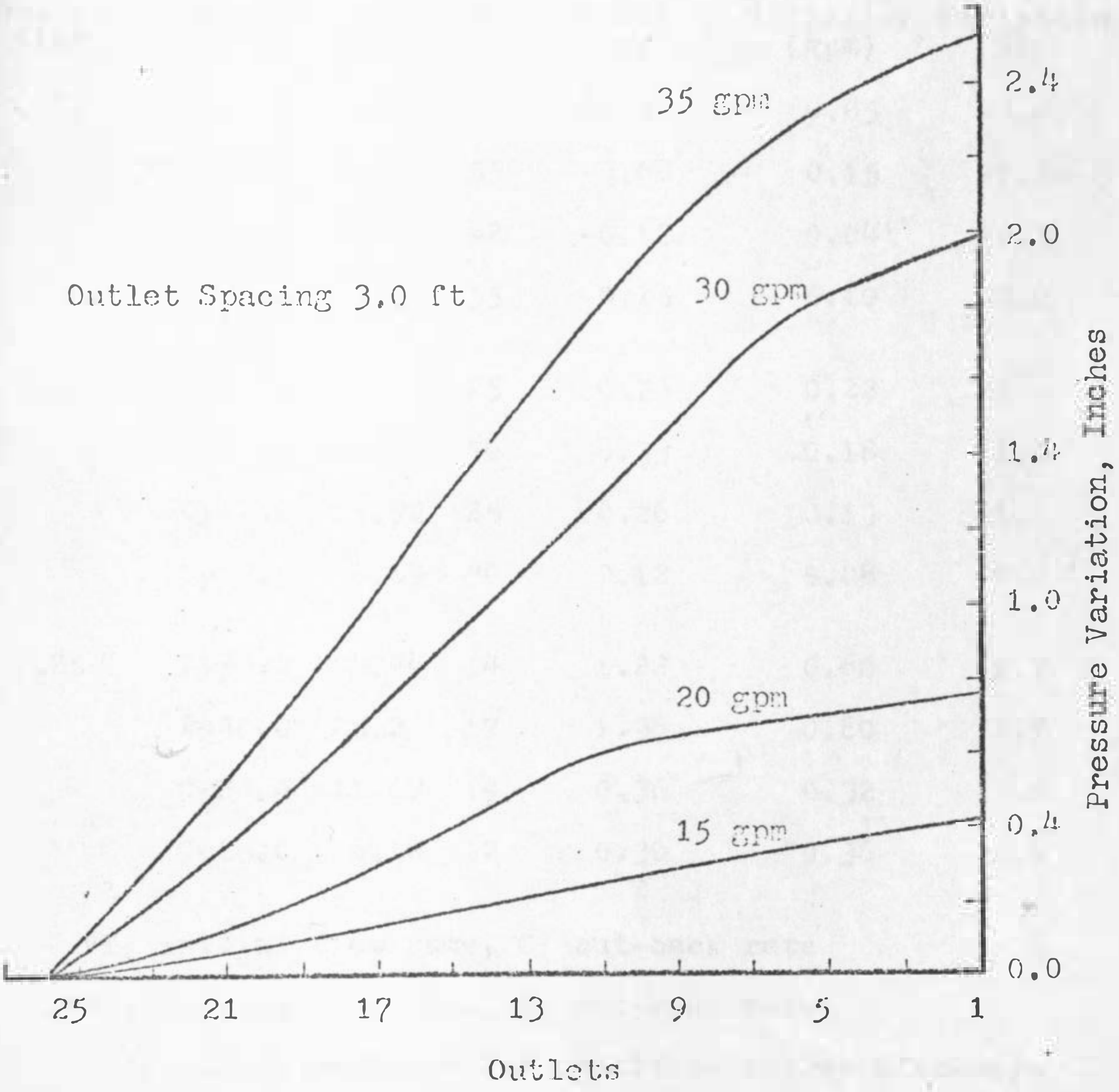

Figure 16. Calculated Pressure Variation for One-Half the Distribution Bank with 1.25-Inch Outlets for 15, 20, 30, and 35 Gallons Per Minute Outlet Discharge

Table 5. Calculated Discharge Variation for One-Half the Maximum Length Distribution Bank for a Given Flow

Outlet Size	Outlet Discharge (gpm)	H[1] (in)	L[2]	Maximum Pressure Variation (in)	Discharge Variation (gpm)	Discharge Variation (%)
0.75	*I_1 12.0	37.17	42	-0.30	0.05	-0.4
	**I_2 9.0	21.00	55	-0.68	0.15	-1.7
	C_1 7.0	12.76	42	-0.13	0.04	-0.5
	C_2 5.0	6.54	55	-0.25	0.10	-2.0
1.00	I_1 20.0	33.67	25	0.76	0.22	1.1
	I_2 17.0	24.20	30	0.53	0.18	1.0
	C_1 12.0	11.92	25	0.26	0.13	1.1
	C_2 9.0	6.64	30	0.12	0.08	0.9
1.25	I_1 35.0	34.44	14	1.22	0.60	1.7
	I_2 30.0	25.2	17	1.36	0.80	2.7
	C_1 20.0	11.07	14	0.36	0.32	1.6
	C_2 15.0	6.18	17	0.30	0.36	2.4

*I_1 initial flow rate, C_1 cut-back rate

**I_2 initial flow rate, C_2 cut-back rate

[1]Pressure required for specified outlet discharge

[2]Maximum number of outlets along one-half the distribution bank for inflow rate 1000 gpm

Table 6. Calculated K_s Values

Test No.	Pressure Differential (ft)	Flow (gpm)	Velocity (fps)	K_s
1	0.192	624.4	4.09	0.277
2	0.142	516.0	3.38	0.311
3	0.092	416.8	2.73	0.285
4	0.067	348.1	2.28	0.295
5	0.042	255.0	1.67	0.330
6	0.017	242.7	1.59	0.146
7	0.0125	187.8	1.23	0.178
8	0.042	106.7	0.70	0.174
		Average K_s =		0.250

DESIGN OF A SYSTEM

The first step in designing the cut-back distribution system is to determine the initial and cut-back flow rates. Matching these two flow rates to a given outlet size and head may require flow rate adjustment. After the flow rates have been determined, the distribution bank lengths can be computed.

Initial Flow Rate Design

The initial flow rate is the flow rate per furrow that will cause the water to advance the length of the furrow in three fourths the time required for infiltration of desired application depth.

$$T_I = 0.75\ T \qquad \text{(Equation 20)}$$

The time for infiltration of the desired application depth can be represented by an equation of the form:

$$T = \frac{60\ D\ (n+1)}{K}^{1/(n+1)} \qquad \text{(Equation 21)}$$

where T = time required for infiltration in minutes,

D = desired depth of application in inches,

and n, k = constants dependent on intake characteristics.

These constants are determined by plotting the intake rate (inches per hour) as a function of time (minutes) on a log-log scale. K equals intake rate in inches per hour at unit time, and n equals the slope of the line.

The advance for flow of water along the furrow can be determined by field testing. The procedure for field testing was outlined by Criddle and others (4). Mathematical methods have been developed to predict the advance rate when field tests cannot be made. One method developed by Wilke and Smerdon (29) predicts the advance rate with an equation of the form:

$$\frac{q\,t}{C\,x} = 1.0 + 0.7165\,\frac{Kt^{a}}{C} \quad \text{when } a = .5 \qquad \text{(Equation 22)}$$

where q = inflow in ft^3 per minute,

t = time in minutes,

C = average area of surface storage in ft^2,

x = distance wetting front had advanced in ft,

and K, a = constants dependent upon infiltration characteristics.

Wilke and Smerdon developed an equation for estimating C of the form:

$$C = 2.75\, d_o^{5/3} \qquad \text{(Equation 23)}$$

where d_o = depth of flow at head of furrow in feet.

Wilke and Smerdon, using data collected, developed an equation for d_o:

$$d_o = 0.075\, (q/s)^{0.5} \qquad \text{(Equation 24)}$$

where q = stream size in ft^3 per minute,

and s = slope of ground surface in percent.

The advance rate for several flow rates can be predicted and plotted. The flow rate that advances the furrow length in 0.75 intake time can be accepted as the initial flow rate.

Cut-back Flow Rate Design

The cut-back flow rate is the flow that matches the intake rate of the soil along the furrow after initial wetting. For design purposes, the cut-back rate used is the rate at which water will infiltrate when the infiltration rate becomes constant. This is expressed mathematically as:

$$q_{CB} = I_C \, S_p \, X/115.5 \qquad \text{(Equation 25)}$$

where q_{CB} = cut-back flow rate in gallons per minute,

I_C = constant intake rate in inches per hour,

S_p = outlet spacing in inches,

and X = field length in feet.

The use of these relationships formsthe basis for a specific design for a field located on the James Valley Research and Extension Center Irrigation Research Farm, Redfield, South Dakota. The design information for the field selected for application of the design criteria and field testing of the distribution bank is as follows:

Water Supply - 1000 gpm delivered at 4 ft of head.

Field Dimensions - Length - 600 ft

Width - 600 ft

Slope - 0.2 percent.

Soil Type - Beotia Silt Loam (12).

Intake Rate (I) = 7.0 $t^{-0.5}$ for 3 ft furrow spacing (5)

where I = intake rate in in per hr,

t = time in min,

From Figure 18 n = -0.5,

and k = 7.0 in per hr.

Intake rate after initial wetting = 0.5 in per hr.

Crop - Corn.

Outlet Spacing - 36 inches.

Desired Application Depth - 3 inches.

Initial Flow Rate

The advance rate can be predicted for initial flow rates of 15, 20, and 25 gallons per minute per outlet using design information based on Wilke's and Smerdon's equations. The predicted advance rates are plotted in Figure 17.

The time for the initial flow rate equals:

$$T_I = 0.75\ T = \frac{60\ D\ (n+1)}{K}^{1/(n+1)}$$

$$= 0.75\ T = (60)(3)\ (-.5+1.0)^{1/(-.5+1.0)}/7$$

$$=(0.75)\ (165)$$

$$= 124 \text{ minutes.}$$

Figure 17 shows that 20 gallons per minute will advance the 600 feet in approximately 120 minutes; therefore,

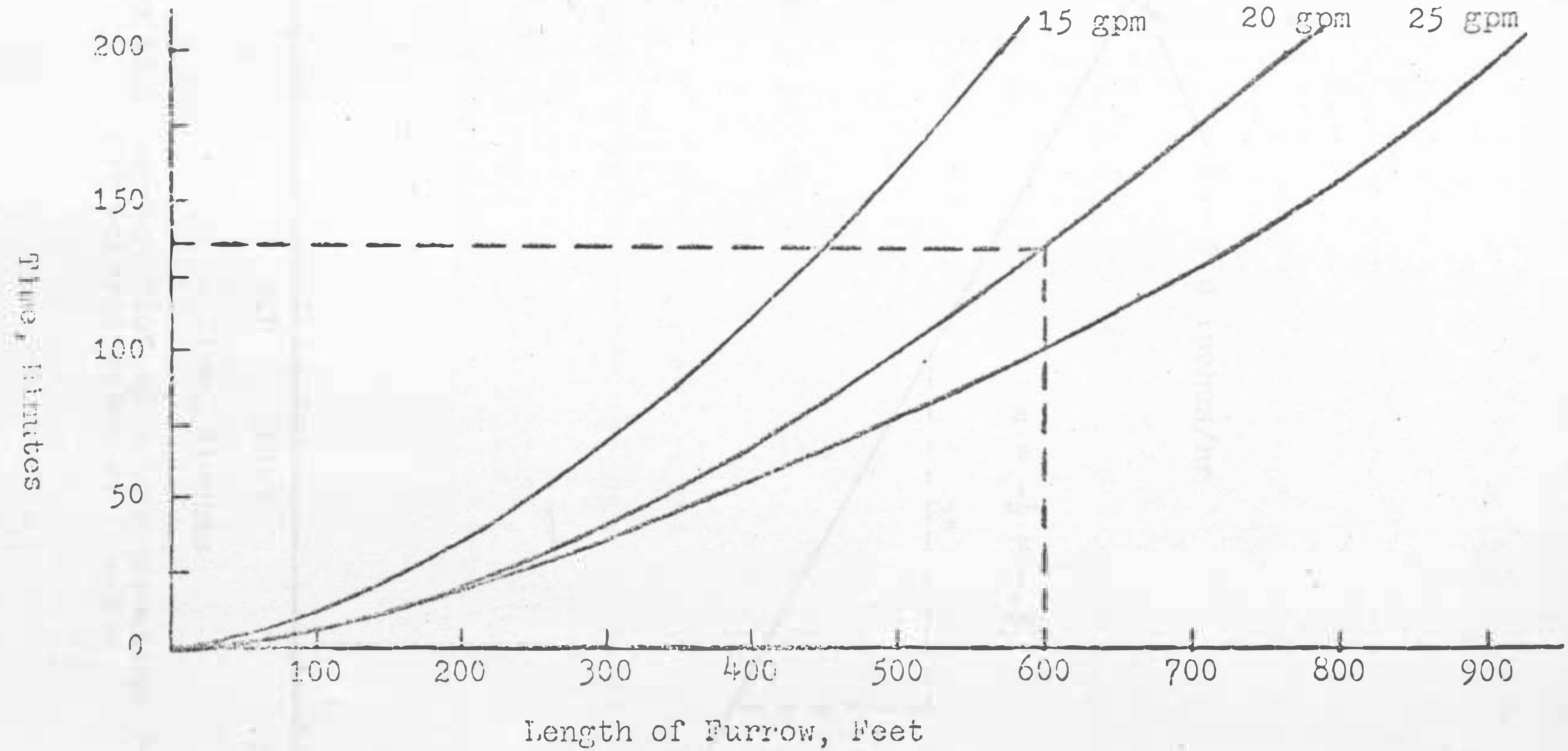

Figure 17. Predicted Rate of Advance Curves for 15, 20, 25 Gallons Per Minute Initial Flow Rates Using Wilke and Smerdon's Equation

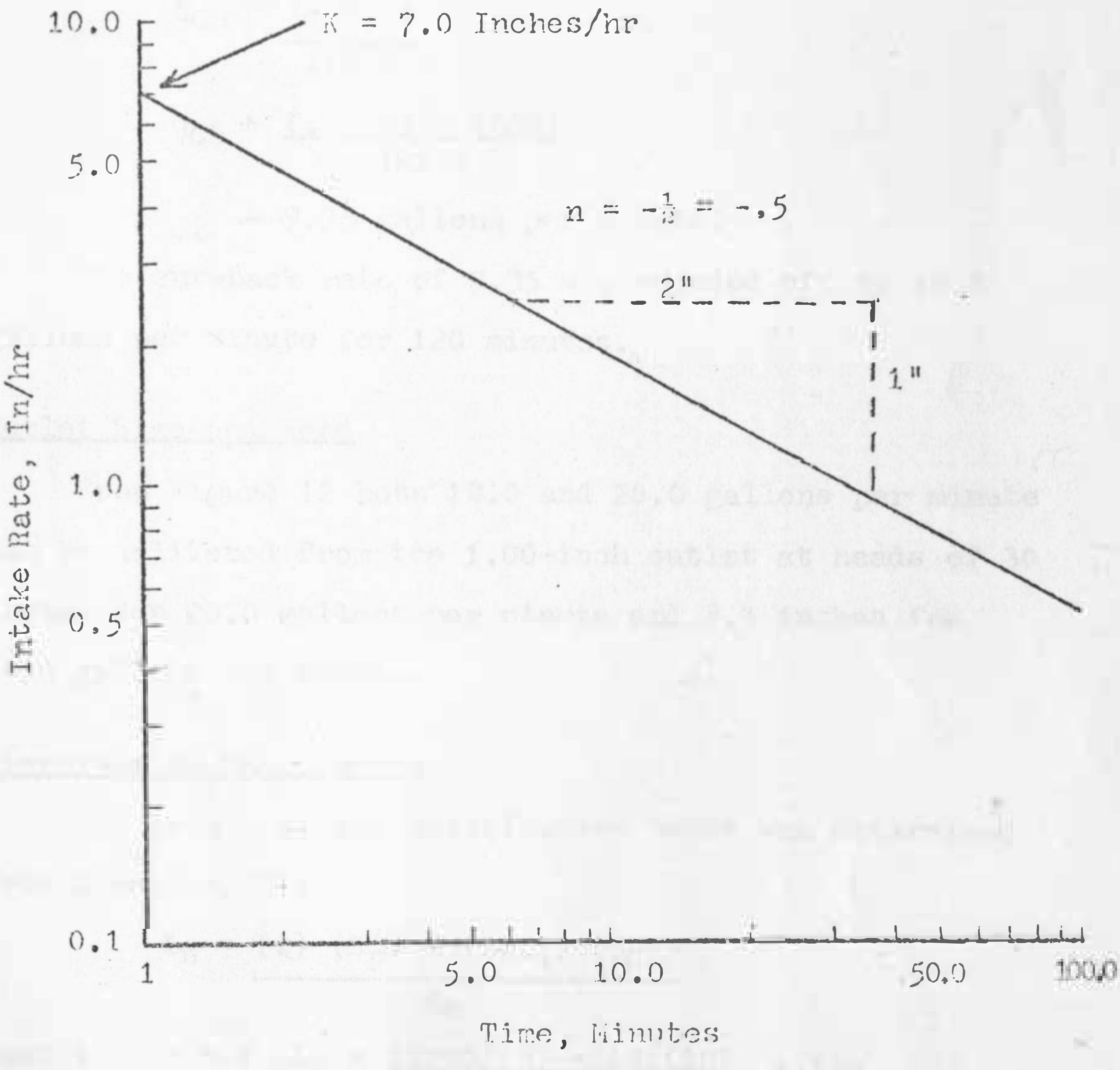

Figure 18. Log-Log Plot of Intake Equation $I = Kt^n$ for Determination of K and n

20 gallons per minute can be used for the initial flow rate.

Cut-back Flow Rate

Equation (25) and design information:

$$q_{CB} = \frac{I_c \; S_p \; X}{115.5}$$

$$q_{CB} = \frac{(.5)\;(36)\;(600)}{115.5}$$

$$= 9.35 \text{ gallons per minute.}$$

The cut-back rate of 9.35 was rounded off to 10.0 gallons per minute for 120 minutes.

Outlet Size and Head

From Figure 12 both 10.0 and 20.0 gallons per minute can be delivered from the 1.00-inch outlet at heads of 30 inches for 20.0 gallons per minute and 8.4 inches for 10.0 gallons per minute.

Distribution Bank Length

The length of the distribution banks was determined from Equation (3):

$$L_n = \frac{(Q)\;(S_p) - (L_{n-1})\;(q_{CB})}{q_c}$$

Bank 1 $\quad n = 1 \quad L_1 = \frac{(1000)(3) - (0)(10)}{20} = 150$

Bank 2 $\quad n = 2 \quad L_2 = \dfrac{(1000)(3) - (150)(10)}{20} = 75$

Bank 3 $\quad n = 3 \quad L_3 = \dfrac{(1000)(3) - (75)(10)}{20} = 112$

Bank 4 $\quad n = 4 \quad L_4 = \dfrac{(1000)(3) - (112)(10)}{20} = 94$

Bank 5 $\quad n = 5 \quad L_5 = \dfrac{(1000)(3) - (94)(10)}{20} = 103$

The total length of the five banks is 534 feet; therefore, length of the last bank will equal 66 feet.

Bank 6 $\quad n = 6 \quad L_6 = 66$

Total Volume of Water Applied Per Furrow

Initial - 20 gpm for 120 minutes = 2400 gallons

Cut-back - 10 gpm for 120 minutes = 1200 gallons

Total = 3600 gallons

= 481.2 ft^3

Total Volume Needed for 3.0-inch Application Per Furrow

= 450 ft^3

Theoretical Application Efficiency from Equation (1)

= 93.5 percent.

FIELD TESTING PROCEDURE AND RESULTS

The field tests were made to evaluate the operating characteristics of the distribution banks under field conditions and to determine applicability of the design criteria. Two test trials were made on the field used for sample design. The sample design was determined for a furrow spacing of 3.0 feet. The actual field furrow spacing was 2.5 feet, therefore, flow rates were reduced to 17.0 gallons per minute for initial and 9.0 gallons per minute for cut-back.

The field testing apparatus consisted of a horizontal 60-foot distribution bank with 24 one-inch diameter outlets spaced at 2.5-foot intervals to match furrow spacing. Three manometer taps and manometers for pressure measurement were installed along bank opposite outlets 1, 12, 24. Pressure regulation was made at outlet 24. Discharge measurements were made at 7 outlets along the bank to determine discharge rates for both initial and cut-back flow rates and discharge variation along the banks. Results of discharge rate measurements and discharge variation are shown in Table 7. The measured discharge rates were approximately 2 gallons per minute larger than design flow rates for both initial and cut-back flow. This result would show field pressure adjustment may be

Table 7. Distribution Bank Discharge Data from Field Testing

Trial 1

Outlet	Initial Flow Discharge (gpm)	Initial Flow Head (in)	Cut-Back Flow Discharge (gpm)	Cut-Back Flow Head (in)
1	18.70	25.0	10.79	7.5
2	19.66		11.51	
7	20.14		11.51	
12	20.14	23.0	11.51	6.5
13	18.22		10.43	
18	18.70		10.43	
24	18.70	25.0	11.19	7.5
Average	19.12		11.05	
Discharge Variation (%)				
Av. to Design		12.5		22.2
Max. to Av.		5.3		4.1
Min. to Av.		-4.7		-5.6

Trial 2

Outlet	Initial Flow Discharge (gpm)	Initial Flow Head (in)	Cut-Back Flow Discharge (gpm)	Cut-Back Flow Head (in)
1	18.70	23.0	10.43	6.5
2	19.18		11.15	
7	19.18		11.51	
12	19.18	23.0	11.51	6.5
13	18.22		11.51	
18	18.70		10.43	
24	19.18	22.5	10.79	6.5
Average	18.91		11.00	
Discharge Variation (%)				
Av. to Design		11.2		22.2
Max. to Av.		1.4		4.6
Min. to Av.		-3.6		-5.2

needed for delivery of design flow rates. The outlet discharge variation along the bank showed horizontal bank placement did have a discharge variation of less than ± 6 percent from average outlet discharge.

Application efficiency and uniformity of application (uniformity coefficient) determinations were made for each test trial. Five furrows near the center of the distribution bank location for each trial were used as a test section. Moisture samples were taken before and after irrigation along each test section at 0-, 100-, 200-, 300-, 400-, 500-, and 600-foot stations. Samples at each station were taken at 1.0-foot intervals to a depth of 4.0 feet. Inches of water per foot of soil determinations were made from each sample. The soil water determination data are shown in Appendix B. Runoff from the five furrows was measured with a H-flume shown in Figure 22.

The determination results of application efficiency from Equation (1) and uniformity coefficient from Equation (2) for the two test trials are shown in Table 8. The distribution patterns of water gained by cut-back irrigation along the field length for each test trial are shown in Figures 23 and 24.

An evaluation of the undesirable operating characteristics was made. The discharge stream deflection observed

Table 8. Results of Application Efficiency and Uniformity Coefficient Determinations

	Trial 1	Trial 2
Water Applied (ft^3)		
Initial flow rate	1533.5	1515.8
Cut-back flow rate	886.2	882.2
Total	2419.7	2398.0
Volume Stored (ft^3)		
3 ft depth	1783.0	1738.4
4 ft depth	2093.8	2075.9
Runoff (ft^3)	92.0	250.6
Application Efficiency (percent)		
3 ft depth	73.7	72.5
4 ft depth	86.5	86.6
Uniformity Coefficient		
3 ft depth	81.7	81.3
4 ft depth	75.8	79.6

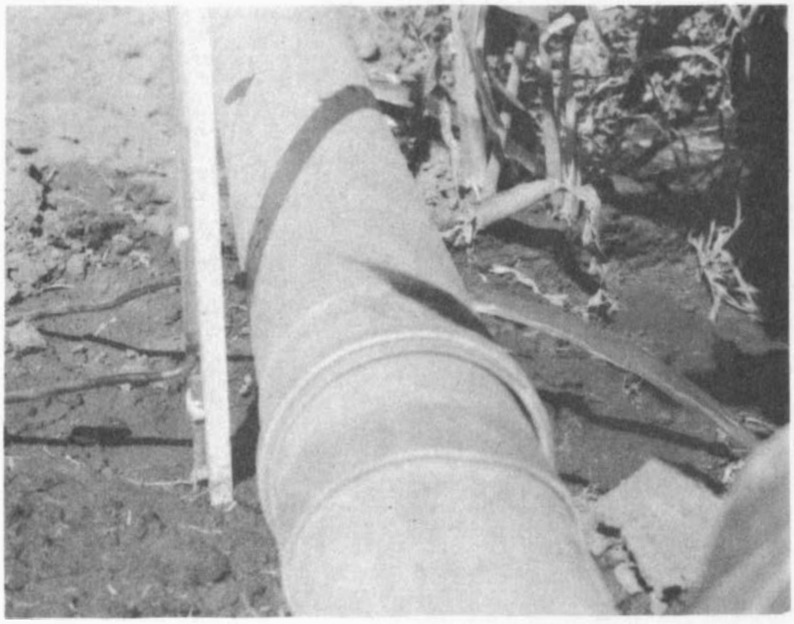

Figure 19. Distribution Bank Discharging Cut-Back Flow Rate

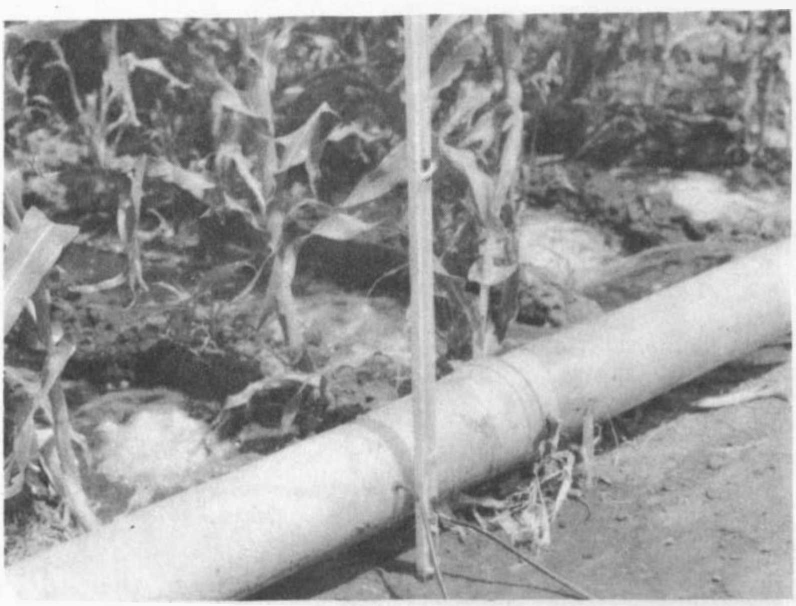

Figure 20. Distribution Bank Discharging Initial Flow Rate

Figure 21. Furrow After Cut-Back Irrigation

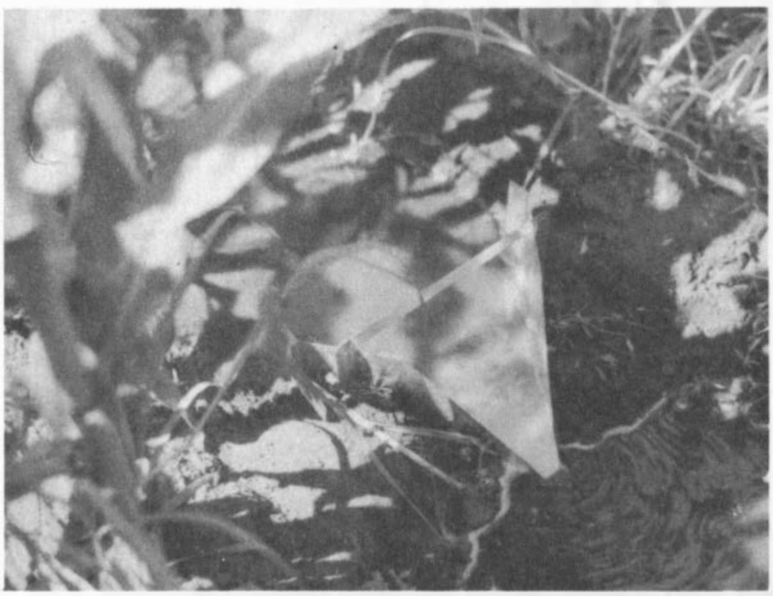

Figure 22. H-Flume Installation

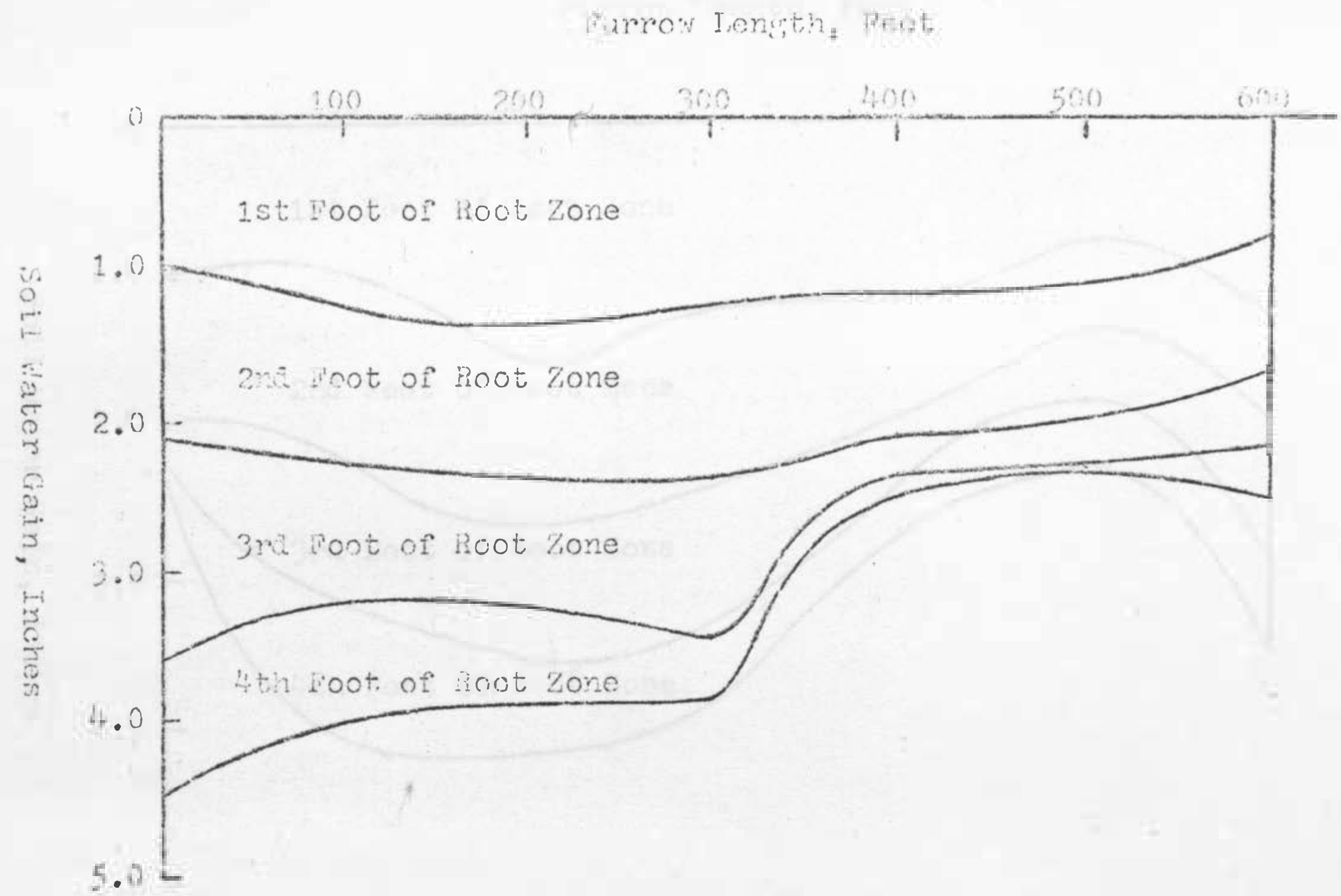

Figure 23. Soil Water Gain by Cut-Back Irrigation Versus Furrow Length, Trial 1

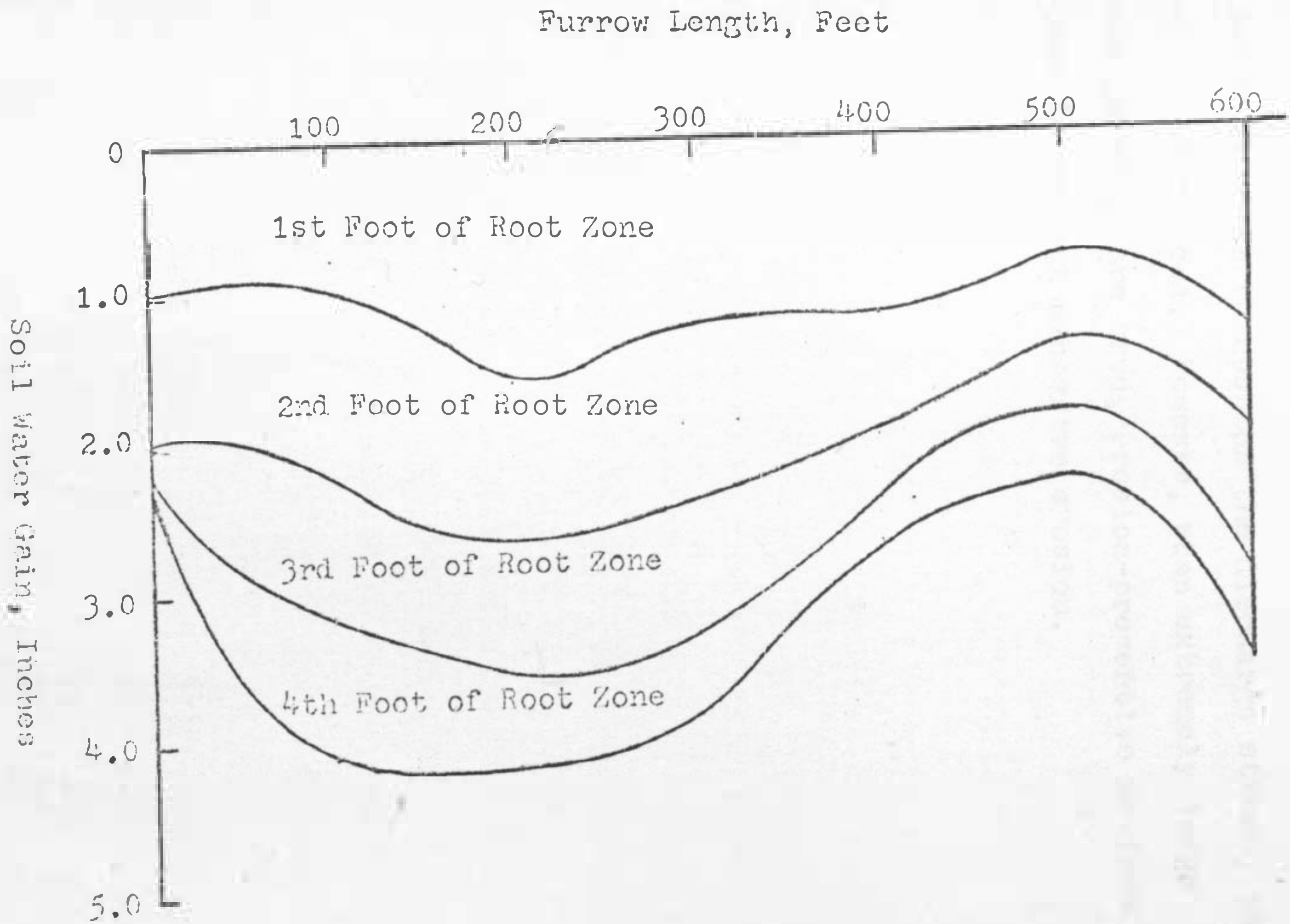

Figure 24. Soil Water Gain by Cut-Back Irrigation Versus Furrow Length, Trial 2

in the field was not large enough to erode ridges between furrows. The erosion potential of discharge stream did not cause excessive erosion. Initially, small holes were eroded by the initial flow rate stream. As these holes filled with water to cushion the discharge stream, the erosion was reduced. However, when extremely large discharge rates are used, erosion-preventive devices should be used to prevent excessive erosion.

SUMMARY AND CONCLUSIONS

The intense competition for available water supplies from domestic, municipal, and industrial consumers and a decreasing labor force available to the irrigator are encouraging the automation of surface irrigation to save labor and increase efficiency.

One system, incorporating both automation and water saving techniques, is the automatic cut-back irrigation procedure. The research undertaken was confined to the design of the distribution portion for an automated closed conduit cut-back furrow irrigation system. The specific purpose was to more accurately define the design components and to field evaluate the resultant design criteria.

The distribution portion was constructed of portable, quick-coupling 8-inch diameter aluminum irrigation pipe with bored, nonregulating outlets for furrow water discharge. This combination allows portability and flexibility while giving the desired means for control of water for discharge of both the initial and cut-back flow rates.

A laboratory study investigated the discharge characteristics of the nonregulating bored outlets. The study determined the factors influencing the uniformity of discharge of outlets equally spaced along the pipe and determined design criteria for distribution bank design.

Results showed the discharge from various outlets as a function of hydraulic head can be represented by an equation of the form: $Q = C\ H^{x}$ where Q is the outlet discharge in gallons per minute, C is the outlet discharge coefficient, H is the head in inches, and x is a constant. Developed equations for the three outlet sizes tested for a head range of 6.0 to 36.0 inches are as follows:

0.75-inch outlet	$Q = 1.940\ H^{0.504}$
1.00-inch outlet	$Q = 3.545\ H^{0.492}$
1.25-inch outlet	$Q = 6.113\ H^{0.493}$

The effect of velocity inside the pipe past the outlet on discharge was evaluated by developing an equation of the form: $Q = C\ H^{x}\ V^{y}$ where Q is the outlet discharge in gallons per minute, C is the outlet discharge coefficient, H is the head in inches, x is a constant, V is the velocity past an outlet in feet per second, and y is a constant. Results show that a velocity from 0.0 to 2.0 feet per second past the outlets at 6.0 and 12.0 inches of head and from 0.0 to 4.0 feet per second past the outlet at 24.0 and 36.0 inches head did not have a discharge variation greater than design limits.

A laboratory evaluation was made to determine the discharge variation from the outlets along the horizontal distribution bank caused by the variation of pressure along the bank. Results showed that for the length of the distribution banks to be used with 0.75, 1.00, and 1.25 inch outlets with an inflow supply rate of 1000 gallons per minute, the discharge variation was less than the design limits set for allowable discharge variation.

The design criteria established were a head discharge relationship for determination of the pressures to be used for discharging the initial and cut-back flow rates. Also, a horizontal placement of the distribution bank should provide for a discharge variation less than the design limits.

A field design using design criteria was completed and field tested. Results show the operating characteristics of the distribution bank to be acceptable for field use. The discharge stream deflection did not appear to be a problem, while the erosion potential of the discharging stream may require installation of erosion control devices when the system is used on highly erodible soils.

The design discharge rates were lower than actual measured discharge rates. The discharge variation along the test bank was within ± 6.0 percent of the average discharge rate or slightly above the design limits.

An application efficiency of 86.6 percent was achieved while the uniformity coefficient was 79.6 for a desired application of 3 inches.

The designed distribution bank for the distribution portion of an automatic cut-back irrigation system fulfills the design criteria. The distribution portion when used with the automatic pressure regulating valves being developed shows promise as a feasible irrigation system.

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APPENDICES

APPENDIX A. Laboratory Data

TABLE A-1

Discharge, Pressure, Velocity Relationships for 0.75-Inch Outlet

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 6-1				
	0	6.0		
Inflow = 48.0 GPM	1	6.0	4.80	0.298
	2	6.0	4.80	0.267
Outflow = 0.0 GPM	3	6.0	4.80	0.236
	4	6.0	4.80	0.204
	5	6.0	4.80	0.173
	6	6.0	4.80	0.141
	7	6.0	4.80	0.110
	8	6.0	4.80	0.079
	9	6.0	4.80	0.047
	10	6.0	4.80	0.016
Test No. 6-2				
	0	6.0		
Inflow = 103.2 GPM	1	6.0	5.05	0.660
	2	6.0	4.80	0.628
Outflow = 55.0 GPM	3	6.0	4.80	0.596
	4	6.0	4.80	0.565
	5	6.0	4.80	0.533
	6	6.0	4.80	0.502
	7	6.0	4.80	0.471
	8	6.0	4.80	0.439
	9	6.0	4.80	0.408
	10	6.0	4.80	0.376
Test No. 6-3				
	0	6.0		
Inflow = 233.4 GPM	1	6.0	4.80	1.513
	2	6.0	4.80	1.482
Outflow = 187.1 GPM	3	6.0	4.80	1.450
	4	6.0	4.56	1.420
	5	6.0	4.56	1.390
	6	6.0	4.56	1.360
	7	6.0	4.56	1.330
	8	6.0	4.56	1.300
	9	6.0	4.56	1.271
	10	6.0	4.56	1.241

TABLE A-1 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 6-4				
	0	6.0		
Inflow = 318.6 GPM	1	6.0	4.80	2.071
	2	6.0	4.80	2.040
Outflow = 271.6 GPM	3	6.0	4.80	2.009
	4	6.0	4.80	1.977
	5	6.0	4.80	1.946
	6	6.0	4.80	1.914
	7	6.0	4.56	1.884
	8	6.0	4.56	1.854
	9	6.0	4.56	1.824
	10	6.0	4.56	1.794
Test No. 12-1				
	0	12.0		
Inflow = 67.4 GPM	1	12.0	6.95	0.418
	2	12.0	6.71	0.374
Outflow = 0.0 GPM	3	12.0	6.71	0.330
	4	12.0	6.71	0.286
	5	12.0	6.71	0.242
	6	12.0	6.71	0.198
	7	12.0	6.71	0.154
	8	12.0	6.71	0.110
	9	12.0	6.71	0.066
	10	12.0	6.71	0.022
Test No. 12-2				
	0	12.0		
Inflow = 190.7 GPM	1	12.0	6.71	0.786
	2	12.0	6.71	0.742
Outflow = 123.4 GPM	3	12.0	6.71	0.698
	4	12.0	6.71	0.654
	5	12.0	6.71	0.610
	6	12.0	6.71	0.566
	7	12.0	6.71	0.522
	8	12.0	6.47	0.479
	9	12.0	6.71	0.436
	10	12.0	6.47	0.392

TABLE A-1 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 12-3				
	0	12.0		
Inflow = 305.2 GPM	1	12.0	6.95	1.954
	2	12.0	6.71	1.909
Outflow = 239.0 GPM	3	12.0	6.47	1.866
	4	12.0	6.47	1.823
	5	12.0	6.47	1.781
	6	12.0	6.71	1.738
	7	12.0	6.71	1.694
	8	12.0	6.47	1.651
	9	12.0	6.71	1.607
	10	12.0	6.47	1.564
Test No. 12-4				
	0	12.0		
Inflow = 426.4 GPM	1	11.9	6.95	2.770
	2	11.9	6.71	2.725
Outflow = 360.7 GPM	3	11.8	6.71	2.681
	4	11.8	6.47	2.638
	5	11.7	6.47	2.595
	6	11.7	6.47	2.553
	7	11.7	6.47	2.511
	8	11.7	6.47	2.468
	9	11.6	6.71	2.425
	10	11.6	6.23	2.383
Test No. 24-1				
	0	24.0		
Inflow = 96.6 GPM	1	24.0	10.07	0.601
	2	24.0	9.83	0.536
Outflow = 0.0 GPM	3	24.0	9.59	0.471
	4	24.0	9.59	0.408
	5	24.0	9.59	0.345
	6	24.0	9.59	0.283
	7	24.0	9.59	0.220
	8	24.0	9.59	0.157
	9	24.0	9.59	0.092
	10	24.0	9.59	0.031

TABLE A-1 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 24-2				
	0	24.0		
Inflow = 202.0 GPM	1	24.0	9.83	1.291
	2	24.0	9.59	1.227
Outflow = 105.8 GPM	3	24.0	9.59	1.164
	4	24.0	9.59	1.101
	5	24.0	9.59	1.039
	6	24.0	9.59	0.976
	7	24.0	9.59	0.913
	8	24.0	9.59	0.850
	9	24.0	9.59	0.787
	10	24.0	9.59	0.725
Test No. 24-3				
	0	24.0		
Inflow = 345.6 GPM	1	24.0	9.83	2.231
	2	24.0	9.83	2.167
Outflow = 249.2 GPM	3	24.0	9.59	2.103
	4	24.0	9.59	2.040
	5	24.0	9.59	1.978
	6	24.0	9.59	1.915
	7	24.0	9.59	1.852
	8	24.0	9.59	1.789
	9	24.0	9.59	1.727
	10	24.0	9.59	1.664
Test No. 24-4				
	0	24.0		
Inflow = 668.7 GPM	1	23.7	9.83	4.348
	2	23.7	9.59	4.284
Outflow = 574.2 GPM	3	23.5	9.59	4.221
	4	23.4	9.35	4.158
	5	23.3	9.35	4.066
	6	23.3	9.35	4.054
	7	23.2	9.35	3.944
	8	23.2	9.59	3.882
	9	23.1	9.59	3.819
	10	23.0	9.11	3.758

TABLE A-1 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 36-1				
	0	36.0		
Inflow = 117.7 GPM	1	36.0	12.23	0.731
	2	36.0	11.75	0.653
Outflow = 0.0 GPM	3	36.0	11.75	0.576
	4	36.0	11.75	0.499
	5	36.0	11.75	0.422
	6	36.0	11.75	0.345
	7	36.0	11.75	0.268
	8	36.0	11.75	0.191
	9	36.0	11.75	0.114
	10	36.0	11.51	0.038
Test No. 36-2				
	0	36.0		
Inflow = 260.3 GPM	1	36.0	12.23	1.665
	2	36.0	11.98	1.586
Outflow = 140.5 GPM	3	36.0	11.98	1.507
	4	36.1	11.98	1.429
	5	36.1	11.98	1.350
	6	36.1	11.98	1.272
	7	36.1	11.98	1.193
	8	36.1	11.98	1.115
	9	36.1	11.98	1.036
	10	36.2	11.75	0.959
Test No. 36-3				
	0	36.0		
Inflow = 478.0 GPM	1	35.9	12.23	3.091
	2	35.9	11.98	3.011
Outflow = 359.3 GPM	3	35.9	11.98	2.933
	4	35.8	11.75	2.855
	5	35.9	11.75	2.778
	6	35.9	11.98	2.701
	7	35.9	11.75	2.623
	8	35.9	11.75	2.546
	9	35.9	11.98	2.468
	10	35.9	11.51	2.395

TABLE A-1 (Continued)

Test No. 36-4

Inflow = 675.4 GPM

Outflow = 558.2 GPM

Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
0	36.0		
1	35.7	11.98	4.385
2	35.7	11.75	4.307
3	35.6	11.75	4.230
4	35.5	11.75	4.153
5	35.5	11.75	4.076
6	35.5	11.75	3.999
7	35.4	11.51	3.923
8	35.4	11.75	3.847
9	35.3	11.75	3.770
10	35.3	11.51	3.694

TABLE A-2

Discharge, Pressure, Velocity Relationships for 1.00-Inch Outlet

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 6-1				
	0	6.0		
Inflow = 85.8 GPM	1	6.0	8.39	0.537
	2	6.0	8.87	0.480
Outflow = 0.0 GPM	3	6.0	8.87	0.422
	4	6.0	8.63	0.364
	5	6.0	8.63	0.307
	6	6.0	8.39	0.252
	7	6.0	8.87	0.194
	8	6.0	8.39	0.138
	9	6.0	8.39	0.082
	10	6.0	8.39	0.028
Test No. 6-2				
	0	6.0		
Inflow = 179.5 GPM	1	6.0	8.63	1.147
	2	6.0	8.87	1.090
Outflow = 93.2 GPM	3	6.0	8.87	1.032
	4	6.0	8.63	0.974
	5	6.0	8.63	0.917
	6	6.0	8.39	0.862
	7	6.0	8.87	0.804
	8	6.0	8.39	0.748
	9	6.0	8.63	0.692
	10	6.0	8.39	0.636
Test No. 6-3				
	0	6.0		
Inflow = 251.3 GPM	1	6.0	8.63	1.618
	2	6.0	9.11	1.560
Outflow = 163.8 GPM	3	6.0	8.87	1.500
	4	6.0	8.63	1.443
	5	6.0	8.87	1.385
	6	6.0	8.63	1.327
	7	6.0	8.87	1.269
	8	6.0	8.63	1.211
	9	6.0	8.63	1.155
	10	6.0	8.63	1.098

TABLE A-2 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 6-4				
	0	6.0		
Inflow = 318.7 GPM	1	6.0	8.39	2.060
	2	6.0	8.87	2.003
Outflow = 232.6 GPM	3	6.0	8.87	1.945
	4	6.0	8.63	1.887
	5	6.0	8.63	1.831
	6	6.0	8.39	1.775
	7	6.0	9.11	1.718
	8	6.0	8.39	1.660
	9	6.0	8.39	1.605
	10	6.0	8.39	1.550
Test No. 12-1				
	0	12.0		
Inflow = 119.9 GPM	1	12.0	11.99	0.747
	2	12.0	11.99	0.669
Outflow = 0.0 GPM	3	12.0	11.99	0.590
	4	12.0	11.99	0.512
	5	12.0	11.99	0.433
	6	12.0	11.51	0.356
	7	12.0	12.47	0.278
	8	12.0	11.99	0.198
	9	12.0	11.99	0.119
	10	12.1	11.99	0.039
Test No. 12-2				
	0	12.0		
Inflow = 224.4 GPM	1	12.0	11.99	1.430
	2	12.0	12.47	1.350
Outflow = 101.2 GPM	3	12.0	12.47	1.269
	4	12.0	12.23	1.188
	5	12.1	12.47	1.107
	6	12.1	12.23	1.026
	7	12.1	12.71	0.945
	8	12.1	12.23	0.863
	9	12.1	12.23	0.783
	10	12.2	12.23	0.703

TABLE A-2 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 12-3				
	0	12.0		
Inflow = 332.1 GPM	1	11.9	11.99	2.136
	2	12.0	12.47	2.056
Outflow = 210.8 GPM	3	12.0	12.47	1.975
	4	12.0	11.99	1.894
	5	12.0	11.99	1.815
	6	12.1	11.99	1.737
	7	12.1	12.47	1.657
	8	12.2	11.99	1.577
	9	12.2	11.99	1.498
	10	12.2	11.99	1.420
Test No. 12-4				
	0	12.0		
Inflow = 381.5 GPM	1	12.0	11.75	2.460
	2	12.0	12.47	2.381
Outflow = 259.7 GPM	3	12.0	12.23	2.300
	4	12.0	11.99	2.201
	5	12.0	12.47	2.141
	6	12.1	11.99	2.060
	7	12.1	12.71	1.980
	8	12.1	11.99	1.899
	9	12.1	12.23	1.819
	10	12.1	11.99	1.740
Test No. 24-1				
	0	24.0		
Inflow = 170.7 GPM	1	24.1	16.79	1.074
	2	24.1	17.26	0.963
Outflow = 0.0 GPM	3	24.1	17.26	0.850
	4	24.1	17.26	0.737
	5	24.2	16.79	0.626
	6	24.2	16.79	0.516
	7	24.2	17.26	0.404
	8	24.3	17.26	0.291
	9	24.3	17.26	0.178
	10	24.3	17.26	0.057

TABLE A-2 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 24-2				
	0	24.0		
Inflow = 435.3 GPM	1	24.0	16.79	2.796
	2	24.0	17.26	2.687
Outflow = 263.6 GPM	3	24.0	17.26	2.572
	4	24.1	17.26	2.459
	5	24.1	17.26	2.346
	6	24.2	17.26	2.233
	7	24.3	17.74	2.118
	8	24.3	16.79	2.005
	9	24.4	17.26	1.894
	10	24.4	16.79	1.782
Test No. 24-3				
	0	24.0		
Inflow = 534.1 GPM	1	23.9	16.79	3.443
	2	24.0	17.26	3.332
Outflow = 366.2 GPM	3	24.0	16.79	3.220
	4	24.0	16.79	3.110
	5	24.1	16.79	3.000
	6	24.1	16.31	2.892
	7	24.1	17.26	2.782
	8	24.1	16.79	2.671
	9	24.2	16.79	2.561
	10	24.2	16.31	2.452
Test No. 24-4				
	0	24.0		
Inflow = 570.0 GPM	1	23.9	16.79	3.679
	2	23.9	17.74	3.565
Outflow = 0.0 GPM	3	23.9	17.74	3.449
	4	24.0	17.26	3.335
	5	24.0	17.26	3.222
	6	24.0	17.26	3.109
	7	24.1	17.74	2.994
	8	24.1	17.26	2.879
	9	24.1	17.26	2.766
	10	24.1	17.26	2.653

TABLE A-2 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 36-1				
	0	36.0		
Inflow = 205.7 GPM	1	36.0	20.62	1.279
	2	36.1	21.10	1.143
Outflow = 0.0 GPM	3	36.1	20.62	1.006
	4	36.2	20.62	0.871
	5	36.2	20.62	0.736
	6	36.2	20.14	0.603
	7	36.3	21.62	0.469
	8	36.3	20.14	0.336
	9	36.3	20.62	0.203
	10	36.3	20.62	0.068
Test No. 36-2				
	0	36.0		
Inflow = 345.6 GPM	1	36.0	20.14	2.198
	2	36.1	21.10	2.063
Outflow = 137.5 GPM	3	36.1	21.10	1.925
	4	36.2	21.10	1.786
	5	36.2	21.10	1.648
	6	36.3	20.62	1.512
	7	36.3	21.10	1.375
	8	36.4	20.62	1.238
	9	36.4	20.62	1.103
	10	36.5	20.62	0.968
Test No. 36-3				
	0	36.0		
Inflow = 516.1 GPM	1	35.9	20.14	3.315
	2	36.0	21.10	3.180
Outflow = 308.0 GPM	3	36.0	21.10	3.042
	4	36.1	20.26	2.905
	5	36.2	21.10	2.768
	6	36.2	20.62	2.632
	7	36.3	21.10	2.495
	8	36.4	20.14	2.360
	9	36.5	21.10	2.225
	10	36.6	21.10	2.087

TABLE A-2 (Continued)

Test No. 36-4

Inflow = 619.3 GPM

Outflow = 413.3 GPM

Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
0	36.0		
1	35.8	20.14	3.991
2	35.9	21.10	3.856
3	36.0	21.10	3.717
4	36.0	20.62	3.581
5	36.1	20.62	3.446
6	36.2	20.14	3.312
7	36.3	21.58	3.176
8	36.3	20.14	3.039
9	36.3	20.62	2.906
10	36.4	20.62	2.771

TABLE A-3

Discharge, Pressure, Velocity Relationships for 1.25-Inch Outlet

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 6-1				
	0	6.0		
Inflow = 147.7 GPM	1	6.0	14.63	0.920
	2	6.0	14.39	0.825
Outflow = 0.0 GPM	3	6.0	14.63	0.730
	4	6.0	14.87	0.633
	5	6.0	15.35	0.534
	6	6.0	15.35	0.434
	7	6.0	14.63	0.335
	8	6.0	14.63	0.240
	9	6.0	14.87	0.143
	10	6.0	14.39	0.047
Test No. 6-2				
	0	6.0		
Inflow = 359.0 GPM	1	6.0	14.39	2.305
	2	6.0	14.39	2.210
Outflow = 212.5 GPM	3	6.0	14.39	2.116
	4	6.0	14.87	2.020
	5	6.0	15.11	1.922
	6	6.1	15.35	1.822
	7	6.1	14.39	1.725
	8	6.2	14.39	1.631
	9	6.2	14.87	1.535
	10	6.2	14.39	1.439
Test No. 12-1				
	0	12.0		
Inflow = 209.6 GPM	1	12.1	21.10	1.303
	2	12.1	20.62	1.167
Outflow = 0.0 GPM	3	12.1	20.62	1.032
	4	12.2	21.10	0.895
	5	12.2	21.58	0.755
	6	12.2	21.58	0.614
	7	12.3	20.62	0.476
	8	12.3	20.62	0.341
	9	12.3	21.10	0.204
	10	12.3	20.62	0.068

TABLE A-3 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 12-2				
	0	12.0		
Inflow = 437.6 GPM	1	12.0	20.62	2.799
	2	12.1	20.14	2.665
Outflow = 229.5 GPM	3	12.1	20.14	2.533
	4	12.2	20.62	2.400
	5	12.2	21.58	2.262
	6	12.3	21.10	2.122
	7	12.4	21.10	1.985
	8	12.4	20.62	1.847
	9	12.4	21.10	1.711
	10	12.5	21.10	1.572
Test No. 24-1				
	0	24.0		
Inflow = 297.4 GPM	1	24.1	29.73	1.945
	2	24.2	29.25	1.753
Outflow = o.o GPM	3	24.3	29.25	1.561
	4	24.3	29.73	1.368
	5	24.4	30.21	1.172
	6	24.5	30.21	0.974
	7	24.5	29.25	0.780
	8	24.6	29.73	0.587
	9	24.6	30.21	0.294
	10	24.6	29.73	0.097
Test No. 24-2				
	0	24.0		
Inflow = 628.3 GPM	1	24.0	28.77	4.023
	2	24.1	28.29	3.836
Outflow = 337.2 GPM	3	24.1	28.29	3.651
	4	24.2	29.25	3.468
	5	24.3	29.73	3.285
	6	24.5	29.73	3.090
	7	24.6	28.77	2.904
	8	24.7	28.77	2.718
	9	24.8	29.73	2.532
	10	25.0	29.73	2.337

TABLE A-3 (Continued)

	Outlet	Static Head Inches	Discharge GPM	Velocity Ft./Sec.
Test No. 36-1				
	0	36.0		
Inflow = 360.2 GPM	1	36.1	35.01	2.228
	2	36.2	34.05	2.002
Outflow = 0.0 GPM	3	36.3	35.01	1.776
	4	36.4	35.97	1.545
	5	36.5	36.93	1.305
	6	36.6	36.93	1.063
	7	36.7	35.49	0.826
	8	36.7	35.97	0.592
	9	36.7	36.93	0.353
	10	36.8	35.49	0.116
Test No. 36-2				
	0	36.0		
Inflow = 709.1 GPM	1	36.1	35.49	4.528
	2	36.2	35.01	4.298
Outflow = 356.6 GPM	3	36.3	34.53	4.070
	4	36.4	35.01	3.842
	5	36.5	36.45	3.612
	6	36.7	36.45	3.374
	7	36.9	34.53	3.146
	8	37.1	34.53	2.920
	9	37.3	35.49	2.690
	10	37.5	35.01	2.459

APPENDIX B. Field Data

TABLE B-1

Soil Moisture Field Data - Trial 1

Direction of flow →

Furrow length, ft.

	0	100	200	300	400	500	600	Soil Surface (Soil Depth)
BISWC*	3.98	4.67	4.50	3.91	3.79	3.68	3.82	
AISWC**	4.95	5.91	5.85	5.12	4.93	4.76	4.60	
SWCG***	0.97	1.24	1.35	1.21	1.14	1.08	0.78	1.0 ft.
BISWC*	4.25	4.80	4.50	4.00	3.74	3.59	3.74	
AISWC**	5.41	5.85	5.50	5.17	4.67	4.48	4.62	
SWCG***	1.16	1.05	1.00	1.17	0.93	0.89	0.88	2.0 ft.
BISWC*	4.34	5.03	4.87	4.52	4.36	4.36	4.34	
AISWC**	5.79	5.93	5.53	5.59	4.62	4.66	4.82	
SWCG***	1.45	0.90	0.66	1.07	0.26	0.30	0.48	3.0 ft.
BISWC*	4.54	5.49	5.35	5.40	5.01	4.98	4.92	
AISWC**	5.44	6.30	6.22	5.92	5.10	5.01	5.28	
SWCG***	0.90	0.81	0.87	0.42	0.09	0.03	0.36	4.0 ft.
Total gain, 3.0 ft. depth, in.	3.58	3.19	3.01	3.45	2.33	2.27	2.14	
Total gain, 4.0 ft. depth, in.	4.48	4.00	3.88	3.87	2.42	2.30	2.50	

*Before irrigation soil water content, in./ft.
**After irrigation soil water content, in./ft.
***Soil water content gain, in.

TABLE B-2

Soil Moisture Field Data - Trial 2

Direction of flow →

Furrow length, ft.

	0	100	200	300	400	500	600	Soil Surface
BISWC*	3.84	4.46	4.22	3.75	3.88	3.85	3.49	
AISWC**	4.82	5.41	5.82	4.95	5.09	4.64	4.78	
SWCG***	0.98	0.95	1.60	1.20	1.21	0.79	1.29	
								1.0 ft.
BISWC*	3.97	4.29	4.16	3.96	3.96	3.77	4.08	
AISWC**	4.71	5.51	5.27	5.15	4.74	4.38	4.79	
SWCG***	0.74	1.22	1.11	1.19	0.78	0.61	0.71	
								2.0 ft.
BISWC*	4.19	4.69	4.72	4.62	4.67	4.26	4.03	
AISWC**	4.72	5.67	5.57	5.53	5.11	4.72	4.95	
SWCG***	0.53	0.98	0.85	0.91	0.44	0.46	0.92	
								3.0 ft.
BISWC*	4.56	5.00	5.34	5.20	5.11	4.64	4.63	
AISWC**	4.92	5.96	5.95	5.74	5.38	5.07	5.24	
SWCG***	0.36	0.96	0.61	0.54	0.27	0.43	0.61	
								4.0 ft.
Total gain, 3.0 ft. depth, in.	2.25	3.15	3.56	3.30	2.43	1.86	2.92	
Total gain, 4.0 ft. depth, in.	2.61	4.11	4.17	3.84	2.70	2.29	3.53	

Soil Depth

*Before irrigation soil water content, in./ft.
**After irrigation soil water content, in./ft.
***Soil water content gain, in.